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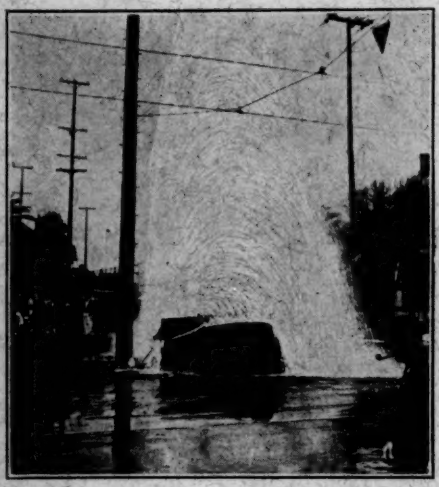

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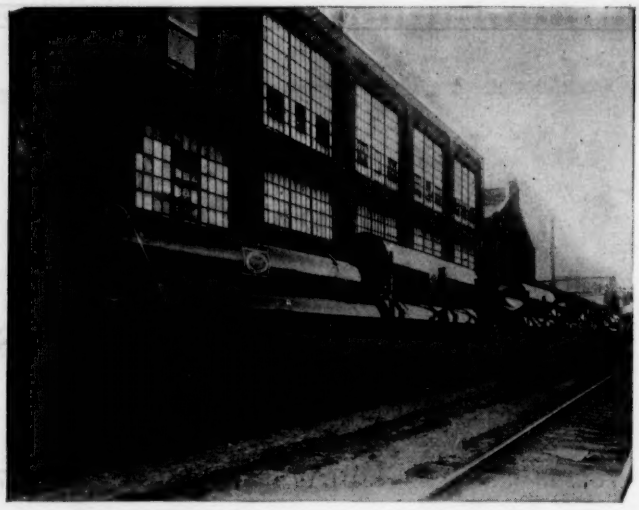

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Vol. 23 JULY, 1931 No. 7

USE OF WATER IN ARID AND SEMI-ARID CITIES[1]

By N. T. Veatch, Jr.[2]

In designing water supply works for cities, the engineer is confronted with the problem of estimating the quantities of water which must be supplied. In solving this problem in arid and semi-arid sections of the country, consideration must be given to many factors which in more humid sections have but little effect upon the demand. Among these factors are the extensive use of water for irrigation purposes, the long periods of dry weather and high temperatures, and the extent of periods during which the use of water is maintained at a high level. In designing distribution systems, an important factor to be considered is that the maximum demand usually occurs in the higher class residential sections, rather than in the business or industrial sections. The relation of demands in several different cities is given in table 1.

Data given in table 1 are believed to be reasonably accurate, as the figures were obtained during detailed studies of the supply and distribution systems. Venturi meter records of usage were available in most cases. Evansville is included in the table for comparative purposes.

From table 1 it is seen that in the strictly arid cities such as Phoenix and Tucson, the average consumption is much greater than in the cities in the humid sections. The best comparison of the relation

[1] Presented before the St. Louis Convention, June 6, 1930.

[2] Consulting Engineer, Kansas City, Mo.

between demands in arid and humid sections can be gotten from the figures for Evansville and Tucson. At the time the data were collected, both cities were served by direct pressure, and both were fairly well metered, Evansville being nearly 100 percent and Tucson slightly over 80 percent. The ratio of maximum hour to maximum day is lower for Tucson because the maximum demands extend over more hours per day and more days per month. The variation between minimum day and maximum day at Tucson is from 3 to something over 10 million gallons which is reflected in the ratio of maximum day to average day. Figure 1 shows the variation between minimum, maximum, and average consumption at Wichita Falls, Texas.

TABLE 1

Showing demands in several cities, having different climatological conditions, gallons per tap

	AVERAGE DAY	MAXIMUM DAY	MAXIMUM HOUR	PERCENT		
				Maximum day to average day	Maximum hour to average day	Maximum hour to maximum day
Evansville	403	540	896	134	222	166
Tucson	916	1,623	2,220	177	225	137
Phoenix	884	1,402	2,386	159	270	170
Wichita Falls	327	578	1,085	177	333	188

As will be shown more clearly in data given later in this paper, it is difficult to draw any definite conclusions from water consumption data from different cities, on account of the fact that no two cities have similar conditions of service. The amount of industrial consumptions differs, the existence and amount of elevated storage varies, and in fact each city presents a separate and distinct problem. However, the demands for water in cities in arid and semi-arid regions present unusual problems, not only in amount of supply necessary, but particularly in regard to the design of distribution systems to meet the demands for irrigation. Figure 2 shows the relation of water consumption to temperature and precipitation in several cities, and shows the comparison between cities in arid and humid regions.

The relation between temperature, precipitation, and pumpage at Tucson, Arizona is shown in figure 3. The effect of both temperature and precipitation upon the pumping rate, is clearly shown.

The demand for irrigation purposes is the main reason for the heavy consumption in arid and semi-arid cities. In connection with an investigation at Tucson, a detailed study of irrigation demands was made. Mr. Richard Bennett, Superintendent, Water Department, prepared scaled plats of some thirty tracts of land in the residential sections, in order to determine the area of lawns and gardens actually

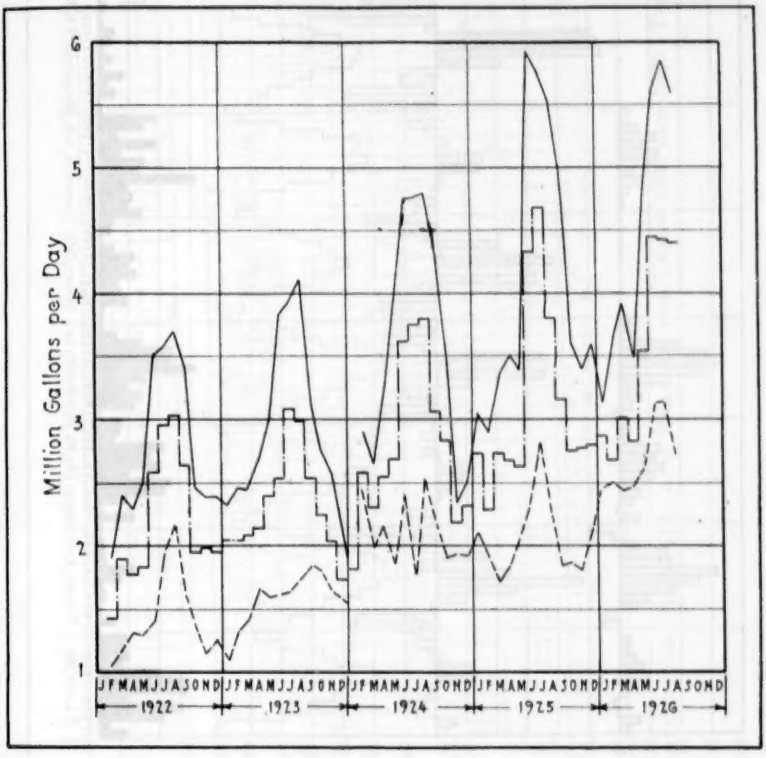

Million Gallons per Day
6
5
4
3
2
1
J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D
1922
1923
1924
1925
1926

FIG. 1. AVERAGE, MAXIMUM AND MINIMUM PUMPAGE—WICHITA FALLS, TEXAS

irrigated and the quantity of water used for this purpose. The entire area of the city was then classified as to the extent of use of water for irrigation. In those sections occupied by the better class residences it was found that approximately 38.5 percent of the total area (including streets and alleys) is actually irrigated and that the average quantity of water used during the maximum month is 30 gallons per 100 square feet of lawn area per day. In those sections

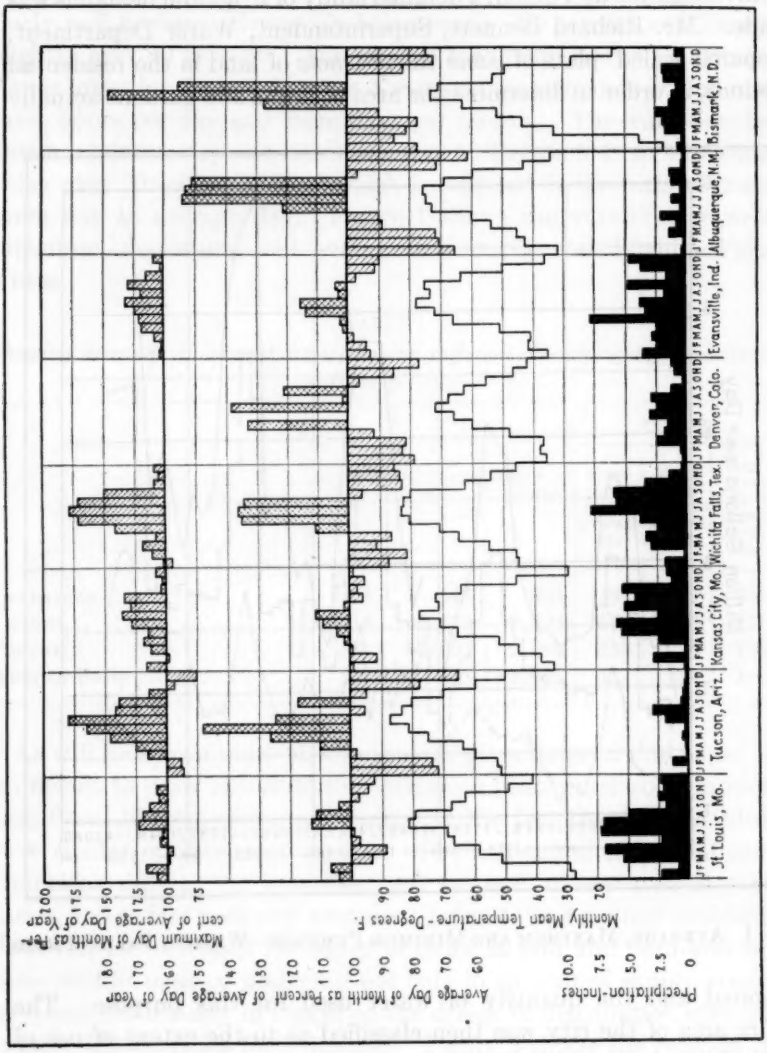


FIG. 2. THE RELATION OF WATER CONSUMPTION TO CLIMATOLOGICAL CONDITIONS

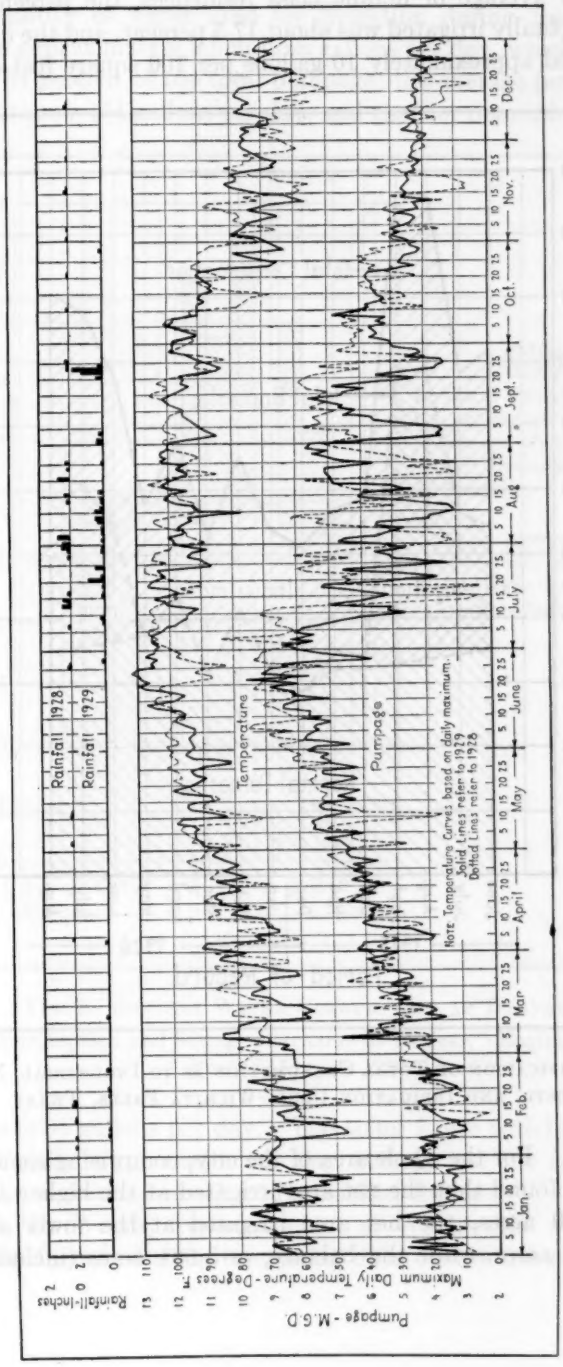


FIG. 3. RELATION BETWEEN TEMPERATURE, RAINFALL, AND PUMPAGE FOR YEARS 1928 AND 1929—TUCSON, ARIZONA

FIG. 2. THE RELATION OF WATER CONSUMPTION TO CLIMATOLOGICAL CONDITIONS

occupied by average or middle class residences, the percentage of total area actually irrigated was about 17.5 percent, and the quantity of water used approximately 10 gallons per 100 square feet of lawn

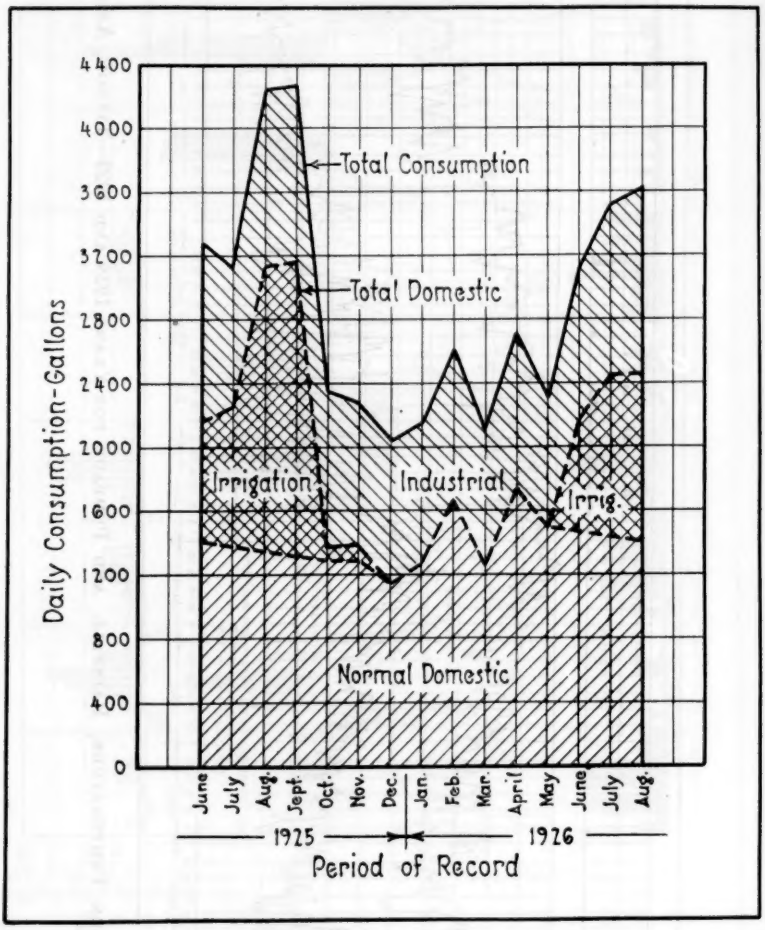


FIG. 4. CLASSIFICATION OF TOTAL CONSUMPTION AS TO INDUSTRIAL, NORMAL DOMESTIC AND IRRIGATION USES—WICHITA FALLS, TEXAS

area per day. For the whole area of the city, comprising some 4,281 acres, it was found that the net area irrigated at the higher average rate was 320 acres, the net area irrigated at the lower average rate was 281 acres, while the balance, or 3,681 acres (including all

streets and alleys), is not irrigated at all. On this basis, then, the total quantity of water used for irrigation during the maximum month is approximately 5.5 million gallons per day, which is approximately 64 percent of the total pumpage; the average per capita use for irrigation is 151 gallons per day, and that for domestic and indus-

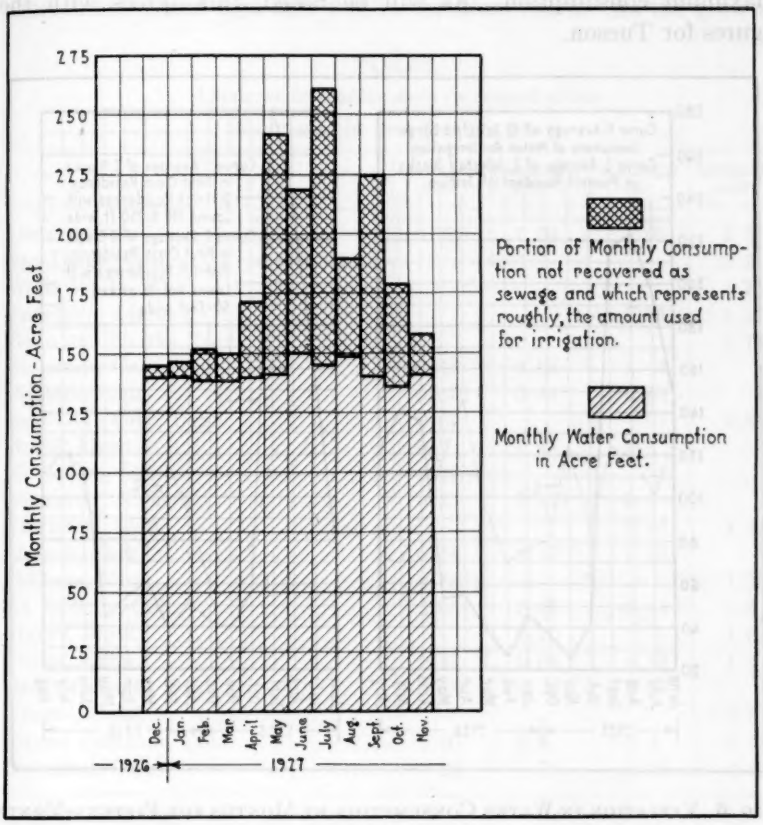


FIG. 5. MONTHLY WATER CONSUMPTION AT DENVER

"Water Consumption and Sewage Discharge at Denver," Engineering News-Record, April 5, 1928

trial uses is 84 gallons per day. The latter figure is to be compared with 101 gallons per capita per day average use for all purposes during the maximum month at Evansville, Indiana.

Mr. A. H. Woods, Superintendent of Waterworks at El Paso, Texas, has furnished some interesting data on the use of water for

irrigation in that city. They indicate that during the maximum month the quantity used for irrigation averages 46 percent of the entire pumpage, and that the average use of water on lawns and gardens that are well maintained is approximately 30 gallons per 100 square feet of area actually irrigated per day during the month of maximum consumption. As will be noted, this agrees with the figures for Tucson.

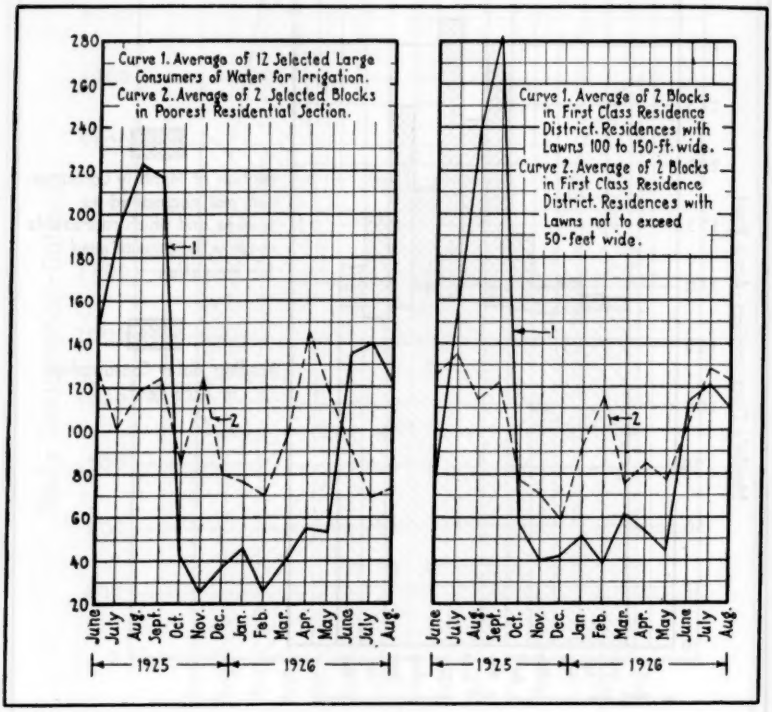

Curve 1. Average of 12 Selected Large Consumers of Water for Irrigation.
Curve 2. Average of 2 Selected Blocks in Poorest Residential Section.
Curve 1. Average of 2 Blocks in First Class Residence District. Residences with Lawns 100 to 150-ft. wide.
Curve 2. Average of 2 Blocks in First Class Residence District. Residences with Lawns not to exceed 50-feet wide.
280
260
240
220
200
180
160
140
120
100
80
60
40
20
June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug.
1925
1926

FIG. 6. VARIATION IN WATER CONSUMPTION BY MONTHS FOR FIFTEEN-MONTH PERIOD ENDING AUGUST 31, 1926—WICHITA FALLS, TEXAS

At Wichita Falls, Texas, the use of water for irrigation during the maximum month, in 1926, averages 58 percent of the total pumpage, although the average per capita quantity so used is but 37 as compared to 151 gallons per day at Tucson. Figures 4 and 5 show the relation, in Wichita Falls, Texas, and Denver, Colorado, of water consumption for irrigation to the total. Figure 6 shows the effect of irrigation of water consumption for several selected customers at Wichita Falls.

Mr. A. H. Douglas, Superintendent Water Department, Wichita Falls, has furnished data from ten selected consumers practicing extensive irrigation, and ten medium users. The figures cover the months of March to September, inclusive, 1929, and indicate an average use for irrigation purposes alone of 1.97 acre feet for the extensive users, and 0.89 acre feet for medium users. As the growing

TABLE 2

Apparent irrigation duty in several cities

Expressed in acre feet per year

	AVERAGE DAY	MAXIMUM MONTH	MAXIMUM DAY	ESTIMATED DUTY
Denver, Colo	0.285	1.28	1.69	2.5
San Diego, Calif				3.1
Walla Walla, Wash				1.71
Pocatello, Idaho			2.34	1.17
Reno and Sparks, Nev				3.75
Stockton, Calif	2.44	4.66	8.44	
Wichita, Kans	0.372	0.44	0.62	
Salt Lake City, Utah	0.67			3.33
Butte, Mont	0.7	1.43	3.5	
El Centro, Calif	0.49			
Bismarck, N. Dak	0.35	0.415	0.9	
Lewiston, Idaho			0.93	1.5
Regina, Sask	0.14	0.17	0.20	
Billings, Mont	0.735*			1.25
Ft. Collins, Colo	2.06†	2.47	3.44	1.5
Harve, Mont		1.75		1.0
Boise, Idaho	0.47	0.66	0.83	1.5
Sioux Falls, S. Dak	0.18	0.20	0.37	
Fresno, Calif				2.5
Grand Junction, Colo	1.60	2.4	2.4	2.25
Trinidad, Colo	1.26	1.75	3.29	2.5

* Five months.

† Six months.

season is largely covered by the period for which data are given, it would appear that an average figure of 2.0 acre feet would be ample to meet the irrigation demands. Wichita Falls is semi-arid, having an average annual rainfall of 28.59 inches. However, the rainfall varies from about 12 inches to 47 inches. The monthly precipitation varies from nothing to 12 inches. Out of 392 months of record in 1926, only 22 had no rainfall. Therefore the above figures on use of

water for irrigation should not be used in cities more strictly classified as arid.

The use of water for irrigation varies, due to other causes than aridity, etc., as shown by the fact that at Yuma, Arizona, practically none is so used, while at El Centro, California, only 30 miles from Yuma, approximately 40 percent of the total pumpage is used for irrigation.

The "duty" of irrigation water used on lawns and gardens or the annual unit quantity required to maintain a lawn or to grow "grass" crops, bears some relation to the distribution of rainfall during the growing season. At Tucson, where the growing season extends for ten months of the year, with practically no rainfall during eight months of this time, well maintained lawns require approximately 84 inches of water per year. At Wichita Falls, where the growing season is eight months, the rainfall is normally quite well distributed through these months, and only some 24 inches of additional water is required for well maintained lawns.

In table 2 is given what had perhaps best be termed "apparent" irrigation duty for several cities, which figures have been obtained from data furnished in questionnaires. These data, in general, expressed the opinion of the Superintendent and were not based on detailed investigation.

At Bisbee, Arizona, a city of 18,000 population, an unusual situation exists, in that a city in an arid section, with an annual average rainfall of approximately 18 inches, has a per capita consumption of only 30 gallons per day. In reply to inquiry as to the reasons for such a situation, Mr. I. A. Rosck, Manager, Arizona Edison Company, gives the following interesting information:

> We note that you question our figures in relation to number of gallons pumped and population served. In this connection I wish to state that we are pumping our water 9 miles against an elevation of more than 1,000 feet, which makes a pressure at our pumps at the station for static and friction head about 600 pounds per square inch. Consequently, as you will note from the enclosed rate card, our water rate is very high, which means that people have to economize on the quantity used. Also as this is a mining community built in among the mountains, we have very few lawns and scarcely no water at all is used for irrigation. We think this will answer the question of the low consumption per capita.
>
> The second question you asked, regarding watering gardens and lawns in Bisbee. As explained above, we have few lawns and gardens. Some irrigation, however, is done by water furnished from one of our mines. This is

principally done in the suburb of Bisbee which is called Warren. Outside of this very little irrigation is done.

We note that you think that the density of population being 29.1 per acre is very dense for an Arizona city. However, the reason for this is that the city of Bisbee originally started among the mountains in a very narrow canyon, and as the city grew it became necessary to crowd houses very close together in order to find enough space for the houses required. This is also the reason we do not have any lawns and gardens.

The rates given are as follows:

1,000 gallons or less	$1.50 minimum
1,010 to 2,000 gallons	2.50 minimum
2,010 to 3,000 gallons	3.25 minimum
Next 22,000 gallons	1.00 per M.
Next 75,000 gallons	0.90 per M.
Next 100,000 gallons	0.85 per M.
Next 100,000 gallons	0.70 per M.
Over 300,000 gallons	0.60 per M.

As a contrast with the situation at Bisbee, can be cited the situation that existed at Laramie, Wyoming, several years ago. At the time of the investigation, there was no direct charge whatever for water, and the consumption was approximately 400 gallons per capita.

The two extreme cases mentioned are good examples of reasons why mass data on water consumption, unless detail studies are made, are not to be relied upon for definite assumptions.

General data have been assembled as to the use of water and factors bearing thereon, in some fifty cities, most of which are located in arid or semi-arid sections. These data are given in tables 3 and 4. One would assume that from this mass of data the existence of some fundamental relationships would be deducible. Of the four factors which would influence the use of water, namely, rainfall, temperature, cost and extent of metering of services, the latter seems to be the most important. Consumption of water has been shown in table 3 in gallons per tap, in order to remove the question of accuracy of population figures, because in a number of cases it was known that the systems served considerable area outside the city proper. The figures are not complete in all columns for some of the cities, due to lack of full data. While no definite relationship seems to be deducible from the figures obtained, except the rather positive effect of metering, it is quite evident that the use of water in arid and semi-arid cities calls for a careful analysis of uses, particularly irrigation, and the distribution of its use over the city.

TABLE 3

Tabulation of data on water consumption

NAME OF CITY	YEAR OF PERIOD	POPULATION	AREA OF CITY, ACRES	CONSUMPTION, MILLION GALLONS							MAXIMUM HOURLY RATE OF CONSUMPTION, M.G.D.	NUMBER OF TAPS	
				Total yearly	Average monthly	Maximum monthly	Minimum monthly	Average daily	Maximum daily	Minimum daily		¾ inch and smaller	1 inch and larger
Reno and Sparks, Nev.	1929	24,000	2,750	3,641.861	303.489	436.172	187.796	9.978	15.300	5.382			
Gunnison, Colo.	1929	1,800		245.408	20.451	26.569	16.747	0.672				423*	
Cheyenne, Wyo.	1929	22,500	3,890	2,926.772	243.600	259.871	216.100	8.105	10.255	7.200		3,262	327
Walla Walla, Wash.	1929	18,000	2,610					5.000	9.000			4,838*	
Ft. Collins, Colo.	1929	10,000	1,778	1,191.250	99.250	178.000	76.000	3.260	8.000	1.000	0.750	3,329	101
Fresno, Calif.	1929	75,000	22,000	8,231.292	685.941	1,272.443	255.932	22.865	46.084	7.779		16,400	2,453
Grand Junction, Colo.	1929	14,000	1,400	1,277.500	105.000	156.000	75.000	3.500	5.048	2.500	0.418	2,600	195
Trinidad, Colo.	1929	15,000	1,600	1,233.816	102.818	149.360	62.874	3.380	7.300	2.602		3,476	60
El Centro, Calif.	1929	10,000	2,000	800.000	60.000	80.000	45.000	2.000	2.700	1.500		1,730	
Salt Lake City, Utah.	1929	137,000	33,589	10,918.560				29.849	42.390	21.650		26,364	1,552
Colorado Springs, Colo.	1929	40,000		2,982.030				8.170	11.546	5.533		14,462*	
Denver, Colo.	1929	348,000	43,520	22,496.820	1,874.735	2,842.830	1,387.520	61.635	116.400	41.750	240.000	65,799	2,035
Pocatello, Idaho	1929	20,000	1,920	1,293.737	107.811	114.788	66.055	3.544	6.237	1.001	0.150	3,800	200
Phoenix, Ariz.	Ends Aug. 1, 1928	55,000						10.000	15.850		27.000	11,309*	
Lewistown, Mont.	1929	6,120	960									1,650	150
Lewiston, Idaho	1929	10,000	2,000	545.130	45.428	79.590	29.920	1.494	2.940			2,330	90
Butte, Mont.	1929	62,500	3,200	3,257.000	271.000	367.000	232.000	9.000	14.500	7.760		11,656	271
Tucson, Ariz.	1928	36,400	4,260	2,111.251	175.940	257.203	119.037	5.768	10.226	2.211	14.000	6,300*	
Billings, Mont.	1929	17,500	1,860	845.529	70.461	122.202	47.484	2.317	5.050	1.450	0.283	3,372	274
Fairbury, Nebr.	1929	6,000	935	290.000	24.150	29.087	15.373	0.795	1.396	0.450		1,533	23
Kansas City, Mo.	April 1, 1929	433,000						52.910	64.600			90,000*	
Rocky Ford, Colo.	1929	5,000	1,600	232.226	19.352	22.777	13.708	0.636	1.082	0.333	1.500	951	12
Edmonton, Alberta,													

									0.636	1.082	0.333	1.500	951	12
Edmonton, Alberta, Canada	1929	74,300	35,840	3,239.450	268.754			8.842	12.000	8.400		3,400	13,600
Austin, Texas	1929	52,000		2,101.060	175.000	265.000	131.000	5.760					
Sioux Falls, S. Dak	1929	35,000	7,342	1,353.587	112.800	137.610	92.115	3.708	5.610	2.420	10.000	6,640	525
San Diego, Calif	1929	168,700		5,924.988	433.749	667.635	306.401	16.233	25.988	6.334		35,622	1,669
Tulsa, Okla	1926	130,000						12.502	18.100			20,000*	
Manhattan, Kans	1929	11,000	1,274	386.405	32.000	38.192	27.348	1.000	2.000	0.800	2.000		
Havre, Mont	1929	7,000	640	246.240	20.502	28.618	15.905	0.648	1.297	0.384	2.160	1,195	337
Hays, Kans	1929	4,800	640	162.414	13.534	20.853	9.401	0.445				825	68
Albuquerque, N. Mex	1926	20,122		472.654	39.368	59.165	25.130	1.295				4,550*	
Minot, N. Dak	1929	16,000	2,400	500.000	41.600	52.000	25.000	1.370	2.000	0.600		1,800	200
San Antonio, Tex	1929	260,000	23,000	8,100.000	670.000	953.000	633.000	22.000	30.000	18.000	56.000	46,552*	
Boise, Idaho	1929	28,000	3,347	850.000	70.833	122.000	46.000	2.323	5.070	1.530	8.000	5,102	465
Yuma, Ariz	1929	8,000	970	241.842	20.153	25.306	14.217	0.663	1.321	0.508			
Dallas, Tex	1929	260,000	18,240	7,824.171	652.014	922.986	520.342	21.436	32.369	15.622		56,253	1,807
Bismarck, N. Dak	1929	10,000	960	269.097	22.200	33.000	17.000	0.737	1.700	0.600	2.500	1,637	63
Wichita Falls, Tex	Aug. 1, 1926	50,000						3.315	5.870		11.050	10,174*	
Salina, Kans	1929	18,000	2,956	460.000	38.333	57.200	29.400	1.260	2.800	0.900	7.920	4,628	376
Wichita, Kans	1929	115,000	9,000	2,931.000	244.250	362.456	188.205	8.000	15.013	5.144	28.000	23,614	397
Waco, Tex	1929	65,000	8,000	1,605.000	133.700	198.560	103.180	4.400	6.550	3.000	12.000	11,600	400
Fargo, N. Dak	1928	35,000	3,202	851.069	71.000	97.000	65.000	2.330	3.500	1.500	14.000	4,900	250
Stockton, Calif	1929	60,000	5,850	1,451.000	120.917	170.247	76.708	3.975	7.583	2.172	16.000	10,150	3,220
Garden City, Kans	1929	6,000	1,280	132.101	11.000	21.062	5.216	0.336	7.000	1.736		921	10
Regina, Sask., Canada	1929	60,000	8,401	1,578.659	129.760	163.027	109.175	4.335	6.119	3.315	9.600	7,900	300
Enid, Okla	1929	33,000		700.000	52.000	90.000	45.000	1.900	3.000	1.500	5.760	6,000*	
Winnipeg, Man., Canada	1929	300,000	14,863	7,362.476	613.546	695.336	663.306	20.171	27.799	16.170			
Oklahoma City, Okla													
Deming, N. Mex	1929	4,000	1,440	77.295	6.441	11.811	3.629	0.215	0.394	0.121	0.394		
Huron, S. Dak	1929	12,000	2,560	231.000		51.000	11.000	0.633	1.900	0.450		2,312	44
Bisbee, Ariz	1929	18,000	620	196.236	16.353	19.800	17.400	0.538	0.860	0.540	0.890	2,500	100
St. Louis, Mo	April 1, 1929	910,000						118.963	146.487			114,974*	

* Includes taps 1 inch and larger.

TABLE 4

Tabulation of data on water consumption

NAME OF CITY	YEAR	CONSUMPTION IN 1,000 GALLONS			NUMBER OF TAPS	RATIO, PER CENT			CONSUMPTION PER TAP, GALLONS			PER CENT METERED	RAINFALL MEAN ANNUAL	MEAN TEMPERATURE FOR YEAR	COST FOR 5 MILLION GALLONS, DOLLARS	ELEVATION, FEET
		Average daily	Maximum daily	Maximum hourly		Maximum day to average day	Maximum hour to average day	Maximum hour to maximum day	Average daily	Maximum daily	Maximum hourly					
Reno and Sparks, Nev.	1929	9,978	15,299			153						0	6.41	51.3	2.20 6.50	4,532
Cunnison, Colo.	1929	672			423				1,588			0				7,670
Cheyenne, Wyo.	1929	8,105	10,255		3,589	126			2,288	2,857		1	18.42	43		6,088
Walla Walla, Wash.	1929		9,000		4,838					1,859					0.90	976
Fort Collins, Colo.	1929	3,260	8,000	18,000	3,430	245	563	226	950	2,330	5,250	8.4	14.08	44.9	1.50	4,985
Fresno, Calif.	1929	22,685	46,084		18,853	205			1,212	2,443		12.6	4.90	63.4	1.20	293
Grand Junction, Colo.	1929	3,500	5,048	10,000	2,795	143	286	200	1,252	1,807	3,574	9.8	10.90	50.4	1.65	4,602
Trinidad, Colo.	1929	3,380	7,300		3,536	216			956	2,064		16.4	19.12	50.2	2.50	5,994
El Centro, Calif.		2,000	2,700		1,730	135			1,157	1,561		0	0.30	71.3	1.90	−50
Salt Lake City, Utah	1929	29,849	42,390		27,916	142			1,068	1,517		95.0	14.47	51.2		4,408
Colorado Springs, Colo.	1929	8,170	11,546		14,462	141			363	798		4.6			1.00	6,098
Denver, Colo.	1929	61,635	116,400	240,000	67,834	178	390	206	908	1,715	3,536		15.42	48.7	0.85	5,283
Pocatello, Idaho	1929	3,544	6,237		4,000	176			886	1,558		78.5			1.00	4,483
Phoenix, Ariz.	To Aug., 1928	10,000	15,860	27,000	11,309	159	270	170	884	1,402	2,386	25.6	6.19	71.2		1,108
Lewistown, Mont.					1,800							6.0	18.26	42.7	1.90	3,974
Lewiston, Idaho	To June 30, 1929	1,494	2,940		2,420	196			618	1,214		100			1.42	757
Butte, Mont.	1929	9,000	14,500		11,927	160			754	1,215		7.6	14.26	43.1	2.75	5,716
Tucson, Ariz.	1928	5,768	10,226	14,000	6,300	177	225	137	916	1,623	2,220	80	7.73	67.2		2,400
Billings, Mont.	1929	2,317	5,050	6,800	3,646	218	294	135	635	1,384	1,864	100	11.96	46.4	1.70	3,115
Fairbury, Nebr.	1929	796	1,396		1,536	176			511	897		99	25.49	52.1	0.75	
Kansas City, Mo.	To April, 1928	52,910	64,600		90,000	122			388	718		80				750
Rocky Ford, Colo.	1929	636	1,082	1,500	963	170	236	138	660	1,124	1,558	100	11.85	51.4	2.80 1.89	4,177
Edmonton, Canada	1929	8,850	12,000		17,000	136			521	706		100	38.07	67.7	1.45	676
Austin, Texas	1929	5,760										100				

Fairbury, Nebr.	1929	796	1,396		1,536	176			511	897		99	25.49	52.1	0.75		3,115	
Kansas City, Mo.	To April, 1928	52,910	64,600		90,000	122			388	718		80					750	
Rocky Ford, Colo.	1929	636	1,082	1,500	963	170	236	138	660	1,124	1,558	100	11.85	51.4	2.80		4,177	
Edmonton, Canada	1929	8,850	12,000		17,000	136			521	706		100			1.89			
Austin, Texas	1929	5,760										100	38.07	67.7	1.45		676	
Sioux Falls, S. Dak.	1929	3,708	5,610	10,000	7,165	152	209	178	517	783	1,396	92.5	34.09	43.7	2.03		1,386	
San Diego, Calif.	To June 30, 1929	16,233	25,988		37,491	160			433	693		100	7.10	60.6	1.85		87	
Tulsa, Okla.	1926	12,502	18,100		20,000	145			625	904							700	
Manhattan, Kans.	1929	1,000	2,000	2,000		200	200	100				100	33.63	53.4			1,030	
Havre, Mont.	To June 30, 1929	648	1,297	2,160	1,532	200	333	157	423	847	1,410	98	10.89	44.5	1.50		2,505	
Hays, Kans.	1929	445			593				498			100	26.57	52.3				
Albuquerque, N. Mex.	1926	1,295			4,550				285				9.21	55.2			5,196	
Minot, N. Dak.		1,370	2,000		2,000	152			685	1,000			9.66	37.5			1,557	
San Antonio, Tex.	1929	22,000	30,000	56,000	46,552	136	254	187	472	644	1,203		29.24	69.0			963	
Boise, Idaho	1929	2,323	5,070	6,000	5,567	218	344	158	417	910	1,437	87.0			1.63		2,739	
Yuma, Ariz.	1929	663	1,321			200						100	0.47	72.9	2.00		140	
Dallas, Tex.	1929	21,436	32,369		58,060	150			389	557		100	28.40	64.6	1.35		512	
Bismarck, N. Dak.	To June 30, 1929	737	1,700	2,500	1,700	231	339	147	433	1,000	1,470	94	16.77	39.0	2.35		1,674	
Wichita Falls, Tex.	To Aug., 1926	3,315	5,870	11,060	10,174	177	333	188	327	578	1,085	100					958	
Salina, Kans.	1929	1,260	2,800	7,960	5,004	222	628	282	252	559	1,584	100	37.81	54.8	1.50		1,200	
Wichita, Kans.	1929	8,000	15,013	28,000	24,011	188	330	186	333	625	1,167	100	34.64	55.0	1.25		1,291	
Waco, Tex.	1929	4,400	6,550	12,000	12,000	149	273	183	367	546	1,000	100	32.91	67.2	1.54		424	
Fargo, N. Dak.	To Sept., 1928	2,330	3,500	8,000	3,150	150	257	171	452	680	1,164	100						
Stockton, Calif.	1929	3,973	7,583	16,000	13,370	191	402	211	297	567	1,197	100	5.92	59.1	1.26		23	
Garden City, Kans.	1929	366	7,000		931	1,910			393	7,520		99	18.30	52.4	0.87		2,836	
Regina, Sask., Canada	1929	4,320	6,100	9,600	8,200	141	222	157	527	744	1,171	97			2.20			
Enid, Okla.	1929	1,900	3,000	5,760	6,000	156	303	192	317	500	960		33.09	58.3			1,269	
Winnipeg, Canada	1929	16,809	23,166			138									1.15			
Oklahoma City, Okla.	1927	7,800	12,260	25,500	21,702	157	327	208	360	564	1,175						1,247	
Deming, N. Mex.	1929	215	394	600		183	279	152				80			2.00		4,325	
Huron, S. Dak.	1929	633	1,900		3,356	300			269	807		94	17.51	43.3	1.25		1,306	
Bisbee, Ariz.	1929	538	860	890	2,600	160	165	103	207	331	342	100	17.78	60.7	4.25		5,350	
St. Louis, Mo.	To April, 1929	118,963	146,487		114,974	123			1,034	1,274		7.7	44.29	55.5			500	
Evansville, Ind.	To May, 1927	8,854	12,050	19,700	21,280	134	222	166	403	540	896	100						

The water superintendents in the cities listed in table 2 have been of much help in preparing this paper, and acknowledgment is made of their courteous assistance, and appreciation expressed therefor.

DISCUSSION

V. M. EHLERS:[3] Mr. Veatch has so ably covered the subject that the writer can only substantiate his statements by citing records in the office of the Texas State Department of Health, and data furnished by city officials and engineers who have been in position to collect valuable information.

Peak consumption of water occurs during the summer months in a number of towns in Texas selected at random. For instance, at Comanche, a town of approximately 4,000 people, the daily consumption varied in 1929 from 38,000 in January to 293,000 gallons in July. Breckenridge with a population of 7,500 showed a daily consumption of 270,000 in January and 315,000 gallons in August. Austin varied from 3 on a January day to 9 million gallons per day in August; Amarillo from 1.8 in January to 6 million in August. Similar variations are noted at Lubbock, Big Spring, and Sweetwater. All the above figures are for 1929.

In making further comparisons it was noted that the consumption for the first months of 1930 shows a decided increase in all the towns quoted. For example, in January, 1929, the average daily consumption at Comanche was 87,000 and for the same month in 1930 it was 162,000 gallons or almost double. There is undoubtedly some increase in the number of water connections in 1930, but there is possibly some real increase in the amount of water consumed per capita.

The towns cited are situated in widely separated sections of the State, and have average rainfalls that vary greatly. Hence, it might be concluded that seasonal variation in temperatures is a most important factor in increasing water consumption. None of these towns, except possibly El Paso, devote any large amount of water to industries. The variations and increases may therefore, in general, be taken to indicate changes in domestic uses from season to season.

Mr. Veatch has stated that metering supplies materially cuts down the amount of water used. We agree with him in this. In arriving at a just comparison of daily consumption rates, however, we would

[3] Chief Sanitary Engineer, State Department of Health, Austin, Tex.

not use metered and unmetered supplies, for in the case of the latter there is undoubtedly much waste. The figures quoted above are all from metered supplies and the water rate remains the same the year round. This would seem to contradict the statement that metering tends to cut down consumption, but, as a matter of fact, the water rate in each of the towns cited is low enough that the consumer is willing to pay an increase in his water bill during the summer to preserve the lawns, shade trees, gardens, etc., and to allow him to take more baths. It would be interesting to collect some data on amounts of water used in various classes of homes, as those with large lawns, those with medium sized yards, and those with no flowers or yards at all. But the conclusions are practically obvious.

RICHARD BENNETT:[4] Mr. Veatch has shown you in his excellent paper some of the unusual conditions with which the City of Tucson

TABLE 5

Water required for fixed lawn sprinklers

AREA, SQUARE FEET	SIZE OF SERVICE, INCHES	APPROXIMATE DEMAND, GALLONS PER MINUTE
2,000	1	50
3,200	1¼	65
4,800	1½	90
8,000	2	150
12,000	2½	200
20,000	3	300

is confronted in its water supply. A new problem confronting water works men in regions similar to Tucson is that of fixed lawn sprinkling systems. The maximum flow demand caused by the extensive use of these systems is increasing to such an extent that in the future it is quite likely that mains will be designed on the sprinkling flow demand rather than on maximum fire flow.

This Department has just been called on for two taps for the sprinkling system of a large hospital which will be 6- and 4-inch sizes respectively and will draw over 1,200 gallons per minute during maximum use. Another recent request was a 2-inch service to supply water for 7,800 square feet of lawn area with a demand of 150 gallons per minute.

The use of fixed sprinkling systems is becoming universal with the

[4] Superintendent, Water Department, Tucson, Ariz.

first class houses in this city. The demand for various size sprinkling areas is shown in table 5.

From table 5 it is seen that merely a few customers can rob a 6-inch main of all flow and pressure. This problem is further complicated by the fact that very little sprinkling is done during the heat of day due to the fact that the hot sun burns the grass after being wetted; therefore, this irrigation load must be supplied after 6 p.m. each evening.

From the foregoing it is seen that the sprinkling flow demand, where extensively used, together with the fire flow demand, will probably render present adequately designed distribution systems inadequate in the future.

EQUITABLE CHARGES FOR SPRINKLER FIRE PROTECTION SERVICE[1]

By L. R. Howson[2]

The question of charges for sprinkler fire protection service is one of the most controversial in the water works rate structure. Although the income from private fire protection in the average water works constitutes only 1 or 2 percent of the total income, quite frequently in rate hearings before regulatory bodies, there is as much evidence introduced relative to sprinkler rates as with respect to all other items of the schedule. It would seem that little controversy should arise over charges for a service which produces savings to the consumers having the service which are usually very much greater than the payments made to the water department which makes the service possible.

Automatic sprinkler fire protection service is of direct value to those making the installation, is often of indirect value to adjoining properties and to a smaller extent, is also of value to the water utility and the community as a whole.

The owner of the property who has installed an automatic sprinkler fire protection system profits through a reduction in his insurance rates usually in amounts varying from 60 to 80 percent of the total premium. To secure this reduction in premium, he is required to expend for the installation of the automatic sprinkler system an amount usually varying between 1 and 4 percent of the building cost. The annual cost to the owner, including therein the fixed charges for interest, depreciation, etc. on the cost of installation, is from 0.1 to 0.5 percent of the building cost. This annual expenditure, as before stated, results in saving from two-thirds to three-fourths of the insurance bill. The cost of installing a sprinkler system is usually repaid in a very few years by reduced insurance premiums.

Adjoining properties benefit from the installation made, due to the fact that the risk of conflagration spreading from the sprinklered

[1]Presented before the Illinois Section meeting, April 28, 1931.

[2]Alvord, Burdick and Howson, Consulting Engineers, Chicago, Ill.

building is lessened and, therefore, the adjoining unprotected property is a better hazard and takes a lower rate.

The water utility profits from automatic sprinkler installations to the extent by which the amount of water used in fighting fires is reduced. The records of the Underwriters for the past thirty years show that over 70 percent of all fires in buildings equipped with automatic sprinklers are put out with less than five sprinkler nozzles going into operation (the equivalent of approximately 90 gallons per minute), and that over 90 percent of all fires in buildings equipped with automatic sprinklers are put out with less than 20 sprinklers in operation, about 350 gallons per minute, the equivalent of two moderate fire streams. This on its face would seem to be a very great reduction in the amount of water used for fire fighting, but when the statistics of cities which keep a record of the amount of water used for fire purposes are studied, it is found that in the average city, only about one-fifth of 1 percent of the total water pumped is used for the purpose of extinguishing fires. The sale value of the water so used in the normal town of 100,000 people having a water works income of $6.00 per capita, is only $1,200 a year. Any saving, therefore, that may be effected by sprinklers in reducing the quantity of water used for fire protection is infinitesimal.

There is further incidental benefit to the community in that industrial fires which are checked at the source with a small spread and small damage, result in fewer interruptions to industrial operations and greater stability of employment and thus benefit the community as a whole.

There are as many varieties and methods of making charges for sprinkler fire protection service as there are plants supplying the service. This lack of uniformity has given rise to much of the difficulty. It is believed that an equitable basis for determining a fair rate for fire protection service must be governed by the fundamental principles governing equitable rates in general, among which are:

a. Every consumer of the utility service should pay his fair share of the total income.

b. No service should be furnished to any consumer at less than cost, for this inevitably results in raising the costs of service to all other consumers.

c. No charge for service should be greater than the value of the service to the user.

d. No charge for service should exceed the cost of securing equal service from another source.

The various costs incidental to the operation of a water utility may be classified under two headings:

1. Those which are incurred due to the utility's ability to meet demands as they arise, more commonly termed "readiness to serve," and
2. Those costs which accrue due to the water actually delivered, commonly called "consumption charges."

The readiness to serve charge must cover all costs such as interest and depreciation, reading of meters, etc. which depend for their amount upon the consumer's ability to take even though for more or less extended periods of time he may be taking no water. The consumption charge must fairly cover the cost of pumping, treating and delivering the water, including all charges which vary in proportion or with respect to the use.

Sprinkler fire protection service is essentially a "readiness to serve" water works function just as public fire protection is. In the ordinary city of 50,000 to 100,000 people, approximately 25 to 35 percent of the total income should equitably be received from the public for fire protection service. For this relatively large percentage of the gross income, the service requires in actual water used only from one-fourth to one-sixth of 1 per cent of the pumpage. It is evident, therefore, that no rate based upon a consumption basis has any real bearing on the problem of fixing a fair charge for municipal fire protection service. The same conclusion applies to sprinkler service.

Automatic sprinkler fire protection service, although effective 90 percent of the time in putting out fires with the use of less water than would probably be required from the public hydrants, nevertheless requires the assistance of the public fire protection system about 10 percent of the time, at which times the sprinkler load is in addition to the demand for the public hydrants, plus the domestic use at the time. At such times, the demand of the sprinkler system clearly increases the maximum demand upon the water system for which the mains, pumps, etc. must be designed. Unless there is some charge for this capacity, the fixed charges on it must be distributed through the rate schedule to the other consumers, thus violating the second fundamental principle of an equitable rate schedule as above stated.

The demand nature of sprinkler fire protection service is apparent when the manner of the operation of the sprinklers is considered. At time of fire in a sprinkler protected property, the nozzles open

without warning and the automatic fire protection system actually appropriates the necessary capacity in pumps, mains, etc. The operation of automatic sprinkler fire protection is under private not public control, is a specialized extra service requiring expenditures by the water departnent in order to supply the service and, therefore, should be paid for by those requiring it. There should be some compensation included to cover inspection costs which experience has demonstrated is necessary to prevent surreptitious use of what should be purely a "readiness to serve" service.

An equitable charge must lie somewhere intermediate between the cost of the service to the water department and its value to the consumer. The cost to the department is represented by a "readiness to serve" charge on the sprinkler connection in the same proportion to the total "readiness to serve" of the entire system as the ability of the sprinkler service to draw water is to the total ability of all services to draw water. Thus if a water system comprises the equivalent of 10,000 $\frac{3}{4}$-inch services, and its total readiness to serve charge is \$50,000 per year, a 4-inch sprinkler service should pay as a minimum approximately $\frac{28}{10{,}000} \times \$50{,}000$, or \$140 per year.

The maximum charge would be indicated by the saving in insurance premium less the fixed charges on the installation cost. Assuming a \$10,000 sprinkler installation in a five-story building 100 feet by 200 feet and served by a single 6-inch connection from the water main with 1,000 sprinkler nozzles and further assuming a saving in insurance cost of \$2,000 per year, the maximum that the consumer could afford to pay the water department would be approximately \$1,000 per year.

Possibly such a large charge would violate fundamental principle (*d*) as the alternative to a connection with the municipal water supply system is the service from an entirely independent source such as a cistern or well with the necessary pumps, tanks, supervision and operation.

The foregoing rather rambling discussion of sprinkler fire protection service and charges for it may be summarized as follows:

1. Automatic sprinkler fire protection service is a specialized service, the control of which is wholly outside of either the utility furnishing the service or the public fire protection system.
2. The ability to furnish adequate sprinkler fire protection service involves costs to the water department.

3. Sprinkler service in common with all other utility services, should at least pay the costs caused by it.
4. Private fire protection service is no more warranted at public expense than private watchmen or other similar services.
5. Fire protection service, including automatic sprinkler protection, is almost entirely a "readiness to serve" function and the charges should be computed as such.
6. Charges for automatic sprinkler fire protection service should be based on the size of the pipe connecting the sprinklers to the water system and not on the number of nozzles installed as is frequently done.
7. While an equitable rate for automatic sprinkler fire protection service is purely a matter of local determination and no specific charge is therefore possible for universal application, it is believed that a fair range of charges probably falls within the following:

Size on connection, inches	*Charge per year, dollars*
2	40– 60
3	75–125
4	125–250
6	250–550

The above range will in general result in payments for automatic sprinkler fire protection service which will remunerate the Water Department for the expenditures involved in furnishing the service and at the same time, enable the protected property to have the protection desired at a cost much less than the saving in insurance premiums. These figures being closer to *cost to the water department* than to the *value of the protection to the private property owner*, should encourage the installation of better fire protection through automatic sprinkler installations.

A SOLUTION OF THE "DUAL FLOW" PROBLEM BY GRAPHICAL ANALYSIS[1]

By Grant K. Palsgrove[2]

The problem of "dual flow," as commonly found in water works practice, constitutes one of three types of primary feeder systems, and provides the highly desirable condition of feed available from two directions in times of heavy draft.

It is the purpose of this paper to show how problems of "dual flow" may be readily solved by the application of graphical analysis leading to a definite result without the usual cumbersome "trial and error" method necessary with an analytical treatment. Incidentally, all graphical methods of analysis have the added advantage of visualizing the relations of the several elements of the problem and in tending to the avoidance of errors.

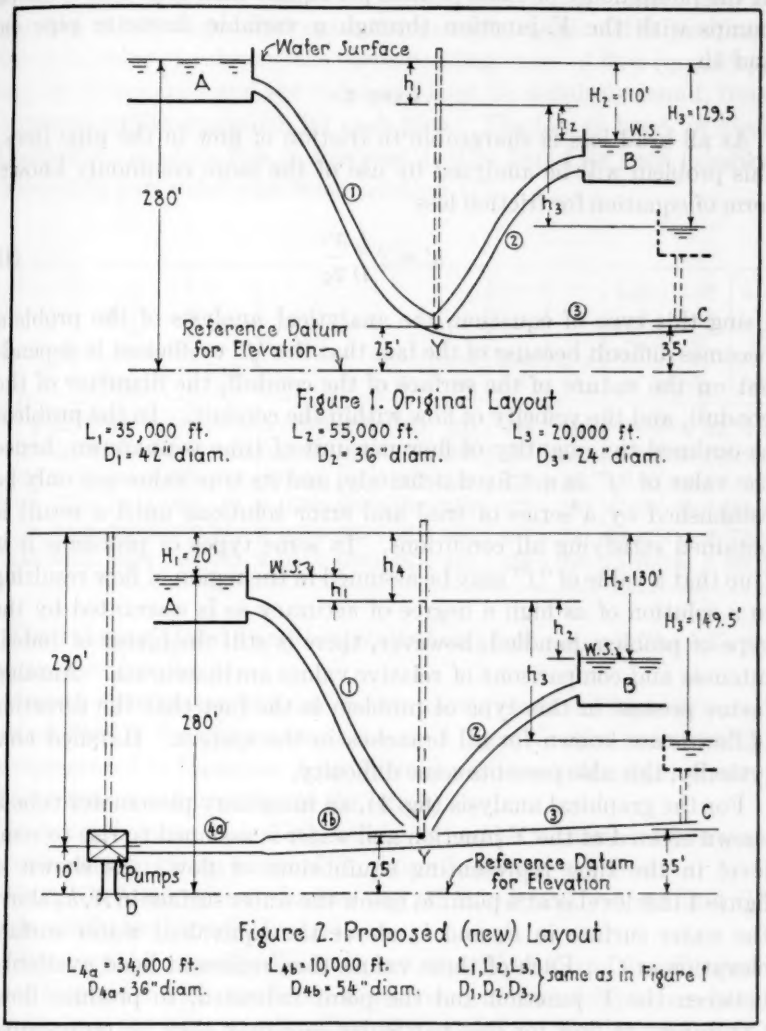
This type of problem as commonly found in practice involves the consideration of what may be termed "*long pipes*," i.e., pipe lines having a length equal to or greater than *one thousand diameters*. For such a line, the drop in pressure (or head) along the line may be charged to the overcoming of frictional resistance to flow in the pipe alone. The error thus involved in neglecting minor losses which may occur in valves, bends, etc., and of disregarding the head utilized in producing velocity will, with such treatment, amount to but a fractional percentage and hence for this type of work may be disregarded. If, however, the problem involves what may be termed a short pipe, it becomes necessary to consider all losses together with velocity head in order to deduce proper results. The graphical analysis, as will be shown later on, is even more readily applied to the short pipe problem than to the one involving long pipes and leads to a definite result which is particularly difficult to obtain with an analytical solution.

Two typical problems will be analyzed in order to illustrate different methods of approach to the graphical solution. Both cases considered will come under the classification known as "long pipes."

[1] Presented before the New York Section meeting, April 22, 1931.

[2] Professor, Hydraulic Engineering, Rensselaer Polytechnic Institute, Troy, N. Y.

1. The problem of three branching mains in a gravity feeder system (fig. 1) composed of a supply reservoir A, a distributing reservoir B, and a feeder main with a pressure of 55 pounds per square inch gage



FIGS. 1–2

at point C. Points A, B, and C are connected at a common junction Y by means of pipe lines 1, 2, and 3, respectively, which have been in service about twenty years.

2. The problem of a proposed additional source of supply D to problem (1) in order to satisfy an increased demand (fig. 2). In this case pumping is required and it is proposed to maintain a pressure at the pump house of 125.5 pounds per square inch gage and connect pumps with the Y junction through a variable diameter pipe 4a and 4b.

CASE I

As all head loss is chargeable to friction of flow in the pipe lines, this problem will be analyzed by use of the more commonly known form of equation for friction loss

$$h' = f \frac{L}{D} \frac{V^2}{2g} \tag{1}$$

Using this type of equation, the analytical analysis of the problem becomes difficult because of the fact that the "f" coefficient is dependent on the nature of the surface of the conduit, the diameter of the conduit, and the velocity of flow within the conduit. In the problem as outlined the quantity of flow per unit of time is unknown, hence the value of "f" is not fixed definitely, and its true value can only be established by a series of trial and error solutions until a result is obtained satisfying all conditions. In some types of problems it is true that a value of "f" may be assumed in the region of flow resulting in a solution of as high a degree of accuracy as is warranted by the type of problem handled; however, there is still the factor of indefiniteness and comparisons of relative values are inaccurate. Another factor present in this type of problem is the fact that the direction of flow is not known for all branches in the system. Handled analytically, this also presents some difficulty.

For the graphical analysis (fig. 1), an imaginary piezometer tube is shown erected at the Y junction and water is assumed to rise to some level in the tube representing equilibrium of flow. As shown in figure 1 this level is at a point h_1 below the water surface in A, h_2 above the water surface in B, and h_3 above the equivalent water surface elevation at C. Each of these values then represent head available, between the Y junction and the point indicated, to produce flow. Conditions of flow for this particular case may then be represented by the following equation

$$Q_1 = Q_2 + Q_3 \tag{2}$$

indicating flow out of A and into B and C.

The next step is to build up a table (table 1), showing the different values of loss of head, h', for corresponding values of flow and with correct values of f. A direct method of accomplishing this is as follows: Assume different values of velocity, the number and value of assumed points dependent upon the possible range of the problem. For each value of velocity, the corresponding value of flow (quantity) may be determined and the value of f may be definitely found, from numerous tables available, for each pipe. The loss of head, or head available to produce flow in this case, is then found for each velocity of flow and particular pipe from equation (1).

TABLE 1

V	Q_1	Q_2	Q_3	$\frac{V^2}{2g}$	f_1	f_2	f_3	$f_1\frac{l_1}{d_1}$	$f_2\frac{l_2}{d_2}$	$f_3\frac{l_3}{d_3}$	$h_1 = f_1\frac{l_1}{d_1}\frac{V^2}{2g}$	$h_2 = f_2\frac{l_2}{d_2}\frac{V^2}{2g}$	$h_3 = f_3\frac{l_3}{d_3}\frac{V^2}{2g}$
1	9.62	7.07	3.14	0.0156	0.026	0.027	0.031	260	495	310	4.06	7.72	4.84
2	19.24	14.14	6.28	0.0621	0.026	0.026	0.030	260	477	300	16.20	29.70	18.60
3	28.86	21.21	9.43	0.140	0.025	0.026	0.030	250	477	300	35.00	66.80	42.00
3.5	33.67	24.75	11.00	0.191	0.025	0.026	0.029	250	477	290	47.80	91.10	55.50
4	38.48	28.28	12.57	0.249	0.025	0.025	0.029	250	459	290	62.30	114.2	72.20
4.5	43.30	31.81	14.14	0.315	0.024	0.025	0.027	240	459	270	75.60	144.8	85.10

The values of Q and h are then plotted on graph paper in the following manner (see fig. 3):

Zero of coördinates of h_1 are taken to represent the highest water surface level in the system, as at *w.s.* in A (fig. 1). Also values of Q, as represented in Equation 2, are plotted positive to the right of zero of coördinates and negative to the left.

In figure 1, when h_1 is 0, Q_1 will be 0; and as h_1 increases in value Q_1 will increase giving curve of Q_1, figure 3, plotted as follows:

From the zero of coördinates lay off h_1 vertically down and Q_1 horizontally to the right from O_1.

Next consider pipe 2. When h_2 is 0, Q_2 will be 0, and h_1 will equal H_2. Hence for this condition a new zero of coördinates has been established, O_2, from which to lay off h_2 and Q_2. The location of O_2 will be at a point below O_1 corresponding to a distance H_2. By Equation 2, a plus Q_2 represents flow into B and as this can only occur when the water surface in piezometer tube at Y is above the *w.s.* in B, the direction of an increase in h_2 for plus Q_2 will be such as to

show a corresponding decrease in h_1. Therefore h_2 is laid off vertically up and Q_2 horizontally to the right from O_2, giving Q_2 curve of figure 3. If flow should be out of B then h_2 would fall below *w.s.* in B and h_2 would be laid off vertically down and Q_2 horizontally to the left from O_2 as shown by broken line in the figure.

Considering pipe 3. When h_3 is 0, Q_3 will be 0, and h_1 will equal H_3. For this condition a new zero of coördinates O_3 is established on the same basis of reasoning as for O_2. From O_3, values of h_3 and Q_3 are laid off similarly to rules set down for h_2 and Q_2, giving Q_3 curve of figure 3.

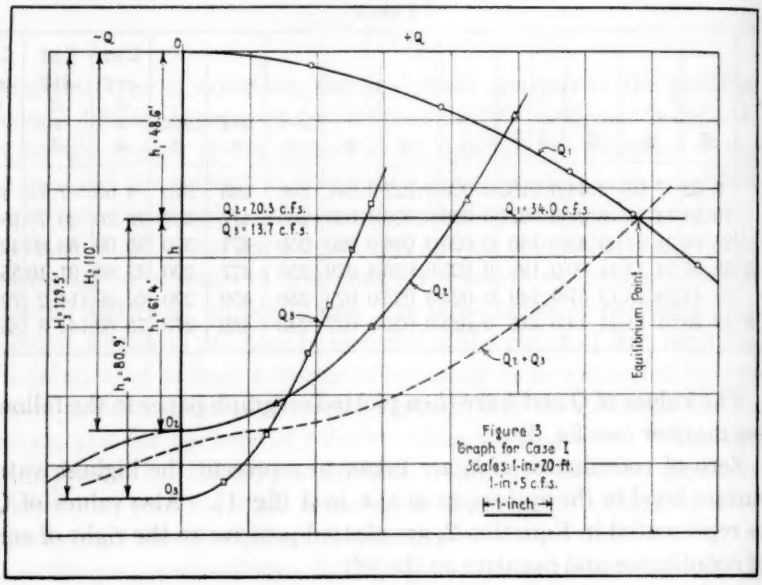


FIG. 3

Having thus established definite relations of all variations of h_1, h_2, and h_3 by shifting the zeros of coördinates vertically, the next step is to satisfy Equation 2. In order to do this, add algebraically Q_2 and Q_3 (horizontally for a particular head) giving curve $(Q_2 + Q_3)$ of figure 3. Where this intersects Q_1 curve, the relation $Q_1 = Q_2 + Q_3$ is satisfied and a definite value of all quantities and directions of flow is established together with the corresponding loss of head in each part of the system. From the latter the hydraulic gradient may be drawn and a study of pressure conditions in the separate parts of the system determined.

The following results are obtained from figure 3 for Case I: $h_1 = 48.6'$, $h_2 = 61.4'$, $h_3 = 80.9'$, $Q_1 = 34.0$ c.f.s., $Q_2 = 20.3$ c.f.s., $Q_3 = 13.7$ c.f.s., and direction of flow is into B as assumed.

It may be noted in passing that this method of analysis lends itself readily to the solution of flow problems in short pipes as all minor losses and velocity head may be expressed as $K\frac{V^2}{2g}$ and simply added to the h term of table 1 for corresponding velocities.

CASE II

In the analysis of this problem, the Hazen-Williams formula

$$V = C\, r^{0.63}\, s^{0.54}\, 0.001^{-0.04} \tag{3}$$

will be used as it has come into such general use in water works practice and its solution is greatly facilitated by employing the

TABLE 2

Q	h'_{4a}	h_{4a}	h'_{4b}	h_{4b}	h_4	h'_1	h_1	h'_2	h_2	h'_3	h_3
7.74						0.092	3.15	0.196	10.78	1.41	28.2
9.28	0.168	5.71	0.023	0.23	5.94	0.129	4.51	0.274	15.10	1.97	39.4
12.38	0.288	9.80	0.040	0.40	10.20	0.220	7.70	0.467	25.70	3.38	67.6
15.47	0.434	14.75	0.060	0.60	15.35	0.332	11.62	0.710	39.10	5.10	102.0
21.66	0.810	27.55	0.113	1.13	28.68	0.620	21.70	1.32	72.55	9.50	190.0
24.76	1.03	35.00	0.144	1.44	36.44	0.800	28.00	1.68	92.45	12.2	244.0
30.94	1.57	53.40	0.218	2.18	55.58	1.21	42.40	2.55	140.4		
40.23	2.55	86.60	0.354	3.54	90.14	1.96	68.60	4.14	228.0		

Note: h' values are per 1,000 feet length of pipe.

Williams and Hazen "Hydraulic Tables" or their special slide rule. The problem as indicated in figure 2 introduces two additional features over that of Case I, i.e., the four branch system and a compound (Series) pipe line. This latter feature adds difficulty in the application of method used for solution of Case I, as Q_{4a} equals Q_{4b} with velocities in the two pipes differing.

Analysis of Case II, while similar to the preceding Case I, will be based on assumed quantities and corresponding head losses found for the several pipe lines making up the system.

The Williams and Hazen Hydraulic Tables are used and a value of C for pipes 1, 2, and 3 taken as 100 while the value of C for pipes 4a

and 4b will be taken as 130 since this part of the system will be new and in good condition.

The determination of factors found in table 2 are self explanatory.

The next step in the solution of this problem is the assumption of the direction of flow. As indicated in figure 2, flow is from D and A toward Y and into B and C from Y. This condition is represented by the equation

$$Q_4 + Q_1 = Q_2 + Q_3 \tag{4}$$

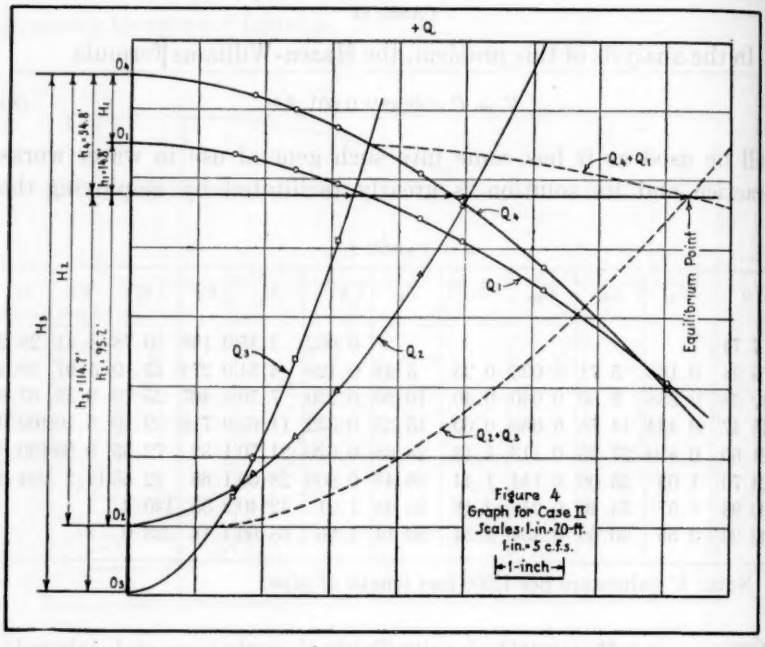


FIG. 4

As in Case I, take the zero of coördinates of h_4 as the highest water surface level, or equivalent water surface level, in the system and lay off h_4, h_1, h_2, h_3 as before establishing new zeros O_1, O_2, O_3 below O_4. Also Q_4, Q_1, Q_2, Q_3 by equation (4) are laid off plus horizontally to the right (fig. 4).

For any particular head, Q_4 and Q_1 curves are added horizontally giving the $(Q_4 + Q_1)$ curve. Likewise the Q_2 and Q_3 curves are added horizontally giving the $(Q_2 + Q_3)$ curve. The intersection of the $(Q_4 + Q_1)$ curve and the $(Q_2 + Q_3)$ curve will then satisfy the relation

expressed by equation (4) and all quantities of flow and loss of head in the several parts of the system have been established.

The following results are obtained from figure 4 for Case II: $h_4 = 34.8'$, $h_1 = 14.8'$, $h_2 = 95.2'$, $h_3 = 114.7'$, $Q_4 = 24.1$ c.f.s., $Q_1 = 17.6$ c.f.s., $Q_2 = 25.2$ c.f.s., $Q_3 = 16.5$ c.f.s., and direction of flow is from D and A towards Y and into B and C from Y, as indicated by assumed equation (4).

If, however, upon the algebraic addition of Q_4 and Q_1 the intersection of the $(Q_4 + Q_1)$ curve and the $(Q_2 + Q_3)$ curve falls above O_1, it would indicate that Q_1 was negative and hence flow would be *into* A, B, and C and out of D; or $Q_4 = Q_1 + Q_2 + Q_3$.

It is to be noted, of course, that there are many variations to this type of problem and that time permits of the solution of only two typical cases as illustrated. The method of attack, however, with possible variations, may be used for any multi-branch, or dual flow, system of feeders and their varying conditions.

TWO FAILURES OF WELDS ON SMALL PATCHES IN LARGE STEEL PIPE[1]

By William W. Brush[2]

The first steel pipe used as part of the delivery system for the water supply of the City of New York was laid as a by-pass around the Ridgewood reservoir, Brooklyn, in 1898. This 60-inch pipe of $\frac{5}{16}$-inch rivetted plate is under only a few feet of head and has required no repairs during its thirty-four years of service. Other steel lines from 48- to 72-inch diameters with rivetted or lock bar longitudinal joints were subsequently installed with satisfactory results as to service, but it was not until about 1925 that the general use of $\frac{1}{2}$-inch plate steel pipe was adopted for delivery (trunk) distribution mains of 30- to 48-inch diameters. At about the same time the development of the electric weld had progressed to the point where it was presented as a competitive process to the rivetted and lock bar types, for longitudinal joints for fabricating large steel pipe. At first, hand welding was advocated and used with a progress rate of twenty-four minutes required to finish a lineal foot of $\frac{1}{2}$-inch plate joint. Now machine welding is general practice with a time rate of four minutes per lineal foot of $\frac{1}{2}$-inch plate joint. The time given to complete a foot of welded joint is that required to secure the ductility and strength specified for the New York pipe. The story of the development of specifications for the electrically welded joint, including tests specified and recent failure of two welded test coupon patches, will be briefly given.

DEVELOPMENT OF SPECIFICATIONS FOR WELDED LONGITUDINAL JOINTS

The Petroleum Iron Works in 1926 urged the New York water department to include in its steel pipe specifications welded joints made by either the gas or electric process and finally agreed that the finished joint should show not less than 90 per cent of the tensile strength of the plate and that the plate with the weld in the center

[1] Presented before the St. Louis Convention, June 3, 1930.

[2] Chief Engineer of Water Supply, Department of Water Supply, Gas and Electricity, New York, N. Y.

could be bent cold in either direction around a $1\frac{1}{2}$-inch pin without showing more than an incipient fracture. The department insisted on both the tensile and bending tests, especially the latter, as it was then and is also now believed that these tests insure a ductile strong welded joint. The coupon for both tensile and bending tests is obtained by the water department inspector who is stationed at the shop selecting the section of joint which looks least satisfactory and having a section about 6- by 14-inch cut out with the joint in the center, the

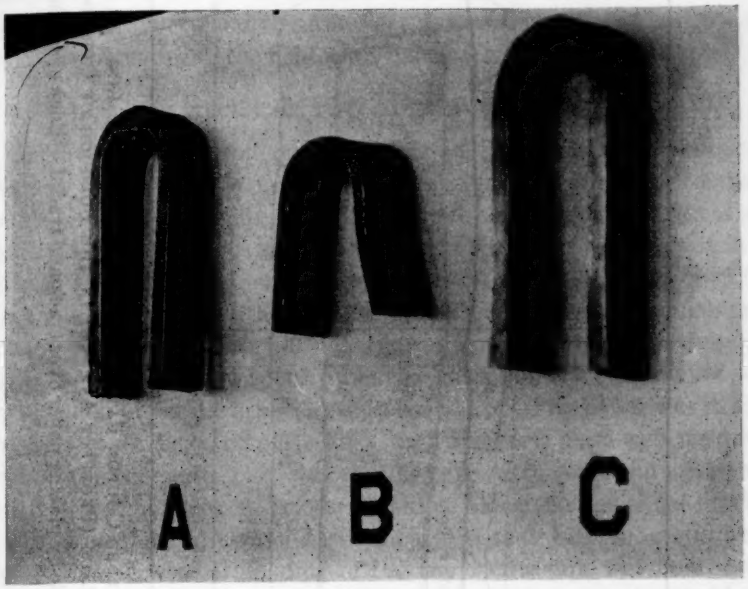


FIG. 1. THREE-TEST COUPONS OF $\frac{1}{2}$-INCH STEEL PLATE BENT COLD THROUGH 180 DEGREES AROUND A $2\frac{1}{4}$-INCH PIN

cut being made with a torch. On the average about one section is cut out of each 15 lengths of pipe and each pipe is about 30 feet in length. In 1925 to 1926 the Petroleum Iron Works fabricated about 13,000 feet of 60-inch pipe with hand made electric welded joints which met the specifications and this line has given no trouble since it was laid. The shop experienced considerable difficulty at first in securing the ductility of joint called for.

As a result of experience in fabricating this pipe line the specifications were raised to 100 percent for strength of joint with

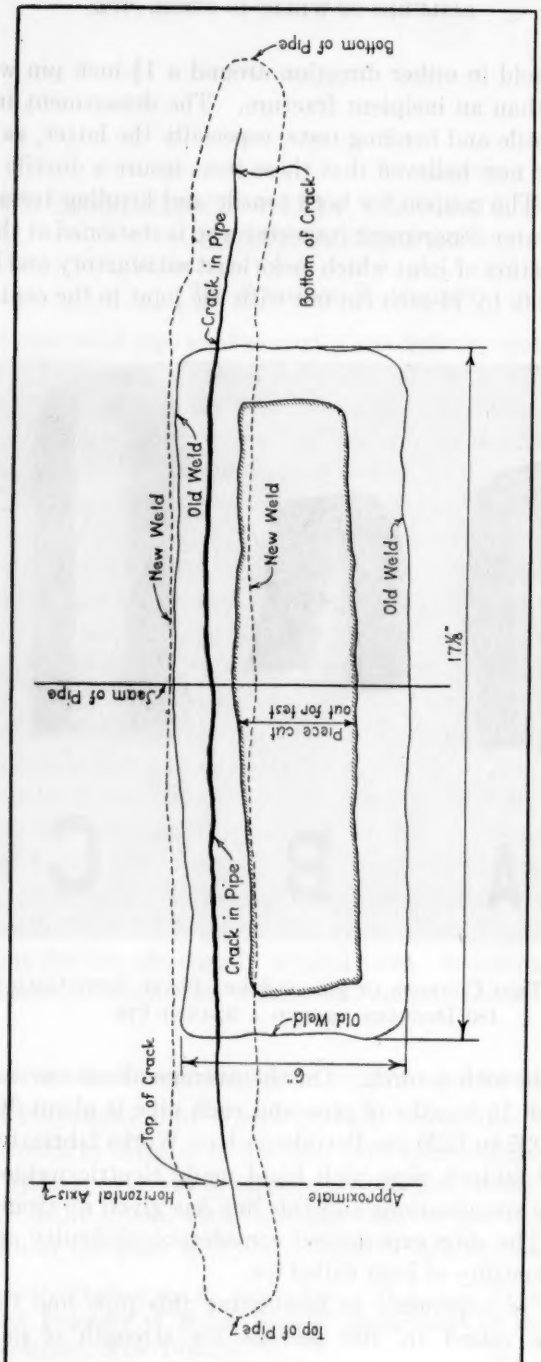


FIG. 2. SKETCH SHOWING CRACK IN WELDED COUPON PATCH ON 60-INCH STEEL PIPE LINE, AT METROPOLITAN AVENUE ABOUT 100 FEET EAST OF 60TH PLACE (PROSPECT AVENUE) IN THE BOROUGH OF QUEENS, NEW YORK CITY

Date: December 15, 1929. Sketch made by placing paper over crack and traced. Edge of paper at right angles to axis of pipe

break occurring outside of the weld and plate immediately adjacent thereto and no sign of fracture of plate or weld when bent cold through 180 degrees around a 2¼-inch pin (fig. 1). These specifications are met by present machine welding, if the welding takes place in an atmosphere from which the air has been largely excluded and if the machine is held down to a relatively slow speed. No difficulty has been experienced with any of the electrically welded longitudinal joints which have been made for different steel pipe contracts for diameters from 30- to 60-inch by the Petroleum Iron Works, the Biggs Boiler Works and the American Locomotive Works and other concerns are entering the field, thus insuring active competition.

FAILURE OF TWO COUPON PATCHES

On December 14 and on December 16, 1929, failure of coupon patches occurred by a sudden rupture along the long side of the patch, the crack extending into the parent metal for several inches at each end of the patch as shown by figure 2. The first failure was in a 60-inch pipe laid in about September, 1928, and which had been in use for about five months under a pressure of about 60 pounds per square inch. The difference in temperature between the time of laying and the failure would probably cause about 9,000 pounds per square inch of tensile stress in the pipe plate. The patch was made by inserting a piece of plate flush with the pipe plate and welding around the four sides. It was repaired by cutting out the weld and cutting into the crack and electric welding along the line of the crack.

The second failure was in a 60-inch pipe that was being laid and due to the accidental flooding of the trench the pipe was floated to a maximum height of about 3 feet. Immediately after the pipe had been brought back to grade and when no work was being done on the pipe in the vicinity of the patch there was a sharp report and it was found the side of the patch had split and the crack extended a few inches into the adjacent plate. This patch was placed on the outside of the pipe and welded on the four outside edges. Repair was effected by cutting out the weld and the crack in the plate and re-welding.

CAUSE OF FAILURE OF COUPON WELDS

There seems to be no question but that the metal in the welds that failed was under high tensile stress as a result of the cooling of the deposited welded metal between two sections of plate that were not free to move.

In the first case the tensile stress set up by the cold water in the pipe plus the initial or cooling stress was in excess of the strength of the weld, which without cooling stress would probably be over 50,000 pounds per square inch. When the fracture occurred the rapidity with which the crack travelled along the 14-inch weld caused an intensity of stress due to shock at the end of the weld that was greater than the strength of the parent plate and the crack extended into the plate until the elasticity of the metal reduced the stress below

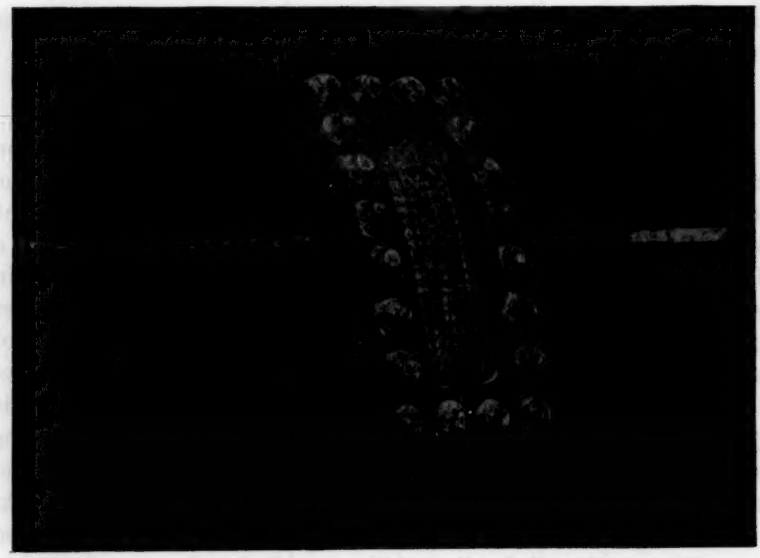


FIG. 3. RIVETED PATCH COVERING OPENING MADE IN REMOVING TEST COUPON FROM 60-INCH PIPE

the plate strength of about 50,000 pounds per square inch. As the crack did not open up more than a small fraction of an inch the amount of water escaping was about equal to the discharge of a 2-inch pipe and did not cause any material damage.

In the second case the movement of the pipe when it floated and when it was later allowed to settle back into place caused some unknown stress in the plate, which together with the cooling stress was greater than the tensile strength and the weld suddenly ruptured extending the crack into the adjoining pipe plate.

COUPON PATCHES CHANGED TO THE RIVETTED FORM

As there appeared to be no practical way of surely avoiding the high cooling stress that would be set up in any welded patch, it was decided hereafter to place the coupon patches on the inside of the pipe and rivet them to the pipe, thus avoiding setting up initial stresses in either the patch or the adjacent pipe plate and this method is now being successfully followed in a 60-inch steel pipe which is being installed (fig. 3).

CONCLUSION

The writer sees no reason to doubt the effectiveness of the welded longitudinal joint which he believes has been well established, provided the joint is made carefully and subjected to tensile and bending tests of coupons cut from the finished joint and also provided that the finished pipe is subjected to heavy sledging along the joints while under a hydrostatic test. He believes, however, that one should not rely on a weld made when the metal must cool between fixed points unless the deposited metal is annealed.

COPPER AND BRASS PIPES AND TUBES[1]

By Wm. G. Schneider[2]

It is the purpose of this paper to present information relating to the use of copper and brass as pipes or tubing for conveying domestic water supplies or for use in wastes, vents or other pipe lines in buildings or around water works.

CORROSION

Rather than enter into an involved technical discussion of this subject it is better to mention briefly a few of the essentials that users of piping should consider when ordering for their requirements.

Bulletin No. 108, Office of the Chief Coördinator of the U. S. Government, states: "All ferrous pipes under certain service conditions rust sooner or later." This rusting or eating away of the body of the material either builds up in the pipe and soon completely or partially clogs it or the rusting action continues until the pipe is eaten through and leaks. The theory of why this is so has been explained time and again and may be found in many textbooks relating to corrosion. Under the circumstances it is very obvious why black ferrous piping is not used and also why galvanized ferrous piping is specified. This means from the standpoint of corrosion that the pipe is just as good as the galvanizing or zinc coating and no better. However, the zinc coating itself is seriously affected by most waters; the exception being where water supplies deposit and form lime coatings on the pipe.

As soon as the zinc coating is pierced in one spot so that the corrosive water can come in contact with the underlying ferrous material and also with the zinc, electrolytic action takes place at a very rapid rate and the zinc coating is soon completely eaten away. The zinc does, however, protect the ferrous material for some time; until it is eaten away. Therefore, when using galvanized piping it is necessary to determine how long the galvanizing will last, because on this depends the life of the piping.

[1] Presented before the Canadian Section meeting, March 12, 1931.

[2] Copper and Brass Research Association, New York, N. Y.

Because of the rusting and rust clogging of ferrous piping these materials are not able to continue to deliver the quantity of water that they do when new. An outstanding piece of research on this subject was recently completed at Harvard University by Fair, Whipple and Hsiao.[3] One set of results compares the quantity of water discharged, starting with new pipes and continuing during the test with the quantity that should be discharged provided no rusting

TABLE 1

Values of $\frac{de}{d}$

MATERIAL	1½-INCH PIPE, COLD WATER	¾-INCH PIPE, COLD WATER	¾-INCH PIPE, HOT WATER, 140°F.
Copper:			
Start of test	1.00	1.03	0.99
End of test	1.00	0.99	1.00
Brass:			
Start of test	1.00	1.04	1.00
End of test	1.00	0.995	1.00
Galvanized steel:			
Start of test	1.00	1.02	0.92
End of test	0.99	0.90	0.89
Galvanized genuine wrought iron:			
Start of test	0.99	0.95	0.91
End of test	1.02	0.91	0.83
Black steel:			
Start of test	1.00	0.99	0.96
End of test	0.66	0.54	0.37
Black genuine wrought iron:			
Start of test	0.98	1.00	0.96
End of test	0.65	0.57	0.31

or clogging had resulted. This is expressed by a relation of $\frac{de}{d}$, meaning the efficiency of the effective diameter to the original diameter.

Briefly summarizing the figures, we note the values of $\frac{de}{d}$ as in table 1.

In other words the copper and brass materials show no change in carrying capacity. The galvanized materials show very little change

[3] Jour. N. E. W. W. A., 44: 4, December, 1930.

when the tests on these pipes were stopped, the reason for stopping being that the galvanizing was gone and from this point on the galvanized pipes would show the same results as the steel and genuine wrought iron pipes that were not galvanized. The black steel and genuine wrought iron pipes showed a tremendous falling off in carrying capacity.

In these tests friction or roughness of the pipes was also measured and is expressed by the value of n. For a smooth pipe, n has a value of 1.70 to 1.75; for rough pipe, n approaches 2.00, but never exceeds it.

TABLE 2

Equivalent sizes of various services

SIZE OF BRASS PIPE	EQUIVALENT SIZE OF IRON PIPE
In hot water lines	
inches	*inches*
½	1
¾	1¼
1	1½
1½	2½
2	3
In cold water lines	
½	¾
¾	1
1	1¼
1½	2
2	2½

These tests were made with water continually in motion. In actual service the water in the pipe is at rest for considerable periods of time. Where it is at rest the corrosive effects are much more severe.

Changes in the copper and brass pipes used in this test "were so slight as to be of little practical importance. The change in n with service can, therefore, be ignored in practice and an average value used." "The average of the values observed during this investigation was 1.754 and the authors (Fair, Whipple and Hsiao) believe that this average can be employed with safety in the calculation of frictional losses." A value of 1.880 is indicated for the galvanized pipes where 2.000 as a value of n should be used for the black steel

TABLE 3

Sizes of copper and brass pipes and tubes

TABLE 3

Sizes of copper and brass pipes and tubes

NOMINAL SIZE	STANDARD PIPE SIZES					COPPER TUBES FOR USE WITH COMPRESSION FITTINGS						EXTRA STRONG	DOUBLE EXTRA STRONG
	Dimensions			Theoretical weight, pounds per foot		Underground			Plumbing				
	Actual outside diameter	Actual inside diameter	Thickness, inches	60 per cent copper 65–68 per cent copper 70 per cent copper, 1 per cent tin	85 per cent copper	Pure copper	Actual outside diameter	Actual inside diameter	Weight, pounds per foot	Actual outside diameter	Actual inside diameter	Weight, pounds per foot	Actual inside diameter	Actual inside diameter
	Actual outside diameter } standard, extra strong, double extra strong													
$\frac{1}{8}$	0.405	0.281	0.0620	0.246	0.253	0.259	.....	.....	.....	.....	.....	.....	0.205	.....
$\frac{1}{4}$	0.540	0.375	0.0825	0.437	0.450	0.460	.....	.....	.....	.....	.....	.....	0.294	.....
$\frac{3}{8}$	0.675	0.494	0.0905	0.612	0.630	0.643	0.500	0.402	0.269	0.500	0.430	0.198	0.421	0.235
$\frac{1}{2}$	0.840	0.625	0.1075	0.911	0.938	0.957	0.625	0.527	0.343	0.625	0.545	0.285	0.542	0.240
$\frac{3}{4}$	1.050	0.822	0.1140	1.24	1.27	1.30	0.875	0.745	0.640	0.875	0.785	0.455	0.736	0.420
1	1.315	1.062	0.1265	1.74	1.79	1.83	1.125	0.995	0.838	1.125	1.025	0.655	0.951	0.585
$1\frac{1}{4}$	1.660	1.368	0.1460	2.56	2.63	2.69	1.375	1.245	1.04	1.375	1.265	0.884	1.272	0.880
$1\frac{1}{2}$	1.900	1.600	0.1500	3.04	3.13	3.20	1.625	1.481	1.36	1.625	1.505	1.14	1.494	1.080
2	2.375	2.062	0.1565	4.02	4.14	4.23	2.125	1.959	2.06	2.125	1.985	1.75	1.933	1.495
$2\frac{1}{2}$	2.875	2.500	0.1875	5.83	6.00	6.14	.....	.....	.....	.....	.....	.....	2.315	1.755
3	3.500	3.062	0.2190	8.31	8.56	8.75	.....	.....	.....	.....	.....	.....	2.892	2.280
$3\frac{1}{2}$	4.000	3.500	0.2500	10.85	11.17	11.41	.....	.....	.....	.....	.....	.....	3.358	2.710
4	4.500	4.000	0.2500	12.29	12.66	12.94	.....	.....	.....	.....	.....	.....	3.818	3.130
5	5.563	5.062	0.2505	15.40	15.85	16.21	.....	.....	.....	.....	.....	.....	4.813	4.063
6	6.625	6.125	0.2500	18.44	18.99	19.41	.....	.....	.....	.....	.....	.....	5.751	5.065
8	8.625	8.000	0.3125	30.05	30.95	31.63	.....	.....	.....	.....	.....	.....	7.625	.....
10	10.750	10.019	0.3655	43.91	45.20	46.22	.....	.....	.....	.....	.....	.....	9.750	.....
12	12.750	12.000	0.375	53.71	55.28	56.51	.....	.....	.....	.....	.....	.....	.....	.....

TABLE 4

General data—sizes and capacity

	NOMINAL SIZE																	
	12	10	8	6	5	4	3½	3	2½	2	1½	1¼	1	¾	½	⅜	¼	⅛
Length of pipe in feet containing one U. S. gallon	0.170	0.244	0.383	0.653	0.957	1.532	2.001	2.614	3.922	5.764	9.578	13.10	21.73	36.27	62.74	100.4	174.3	310.4
Length of pipe in feet containing one cubic foot	1.273	1.822	2.865	4.886	7.158	11.46	14.97	19.55	29.34	43.12	71.65	97.99	162.6	271.3	469.3	751.0	1304.0	2322.0
U. S. gallons contained in one lineal foot of pipe	5.875	4.105	2.611	1.531	1.045	0.653	0.500	0.363	0.255	0.174	0.104	0.076	0.046	0.028	0.016	0.010	0.006	0.003
Pounds of water contained in one lineal foot of pipe	49.03	34.25	21.78	12.77	8.718	5.455	4.168	3.192	2.127	1.447	0.871	0.637	0.384	0.230	0.133	0.083	0.048	0.027
Square feet of outside or radiating surface per lineal foot of pipe	3.338	2.814	2.258	1.734	1.456	1.178	1.047	0.916	0.753	0.622	0.497	0.435	0.344	0.275	0.220	0.177	0.141	0.106
Length of pipe in feet per square foot inside surface	0.318	0.381	0.477	0.624	0.755	0.955	1.091	1.247	1.528	1.852	2.387	2.792	3.597	4.647	6.112	7.732	10.19	13.59
Length of pipe in feet per square foot outside or radiating surface	0.300	0.355	0.443	0.577	0.687	0.849	0.955	1.091	1.329	1.608	2.010	2.301	2.905	3.638	4.547	5.659	7.074	9.431

TABLE 5

Data relating to threads

	NOMINAL SIZE																	
	⅛	¼	⅜	½	¾	1	1¼	1½	2	2½	3	3½	4	5	6	8	10	12
Number of threads per inch of screw	27	18	18	14	14	11½	11½	11½	11½	8	8	8	8	8	8	8	8	8
Number of perfect threads	5.12	5.23	5.34	5.47	5.63	5.85	6.12	6.32	6.69	7.10	7.60	8.00	8.40	9.25	10.10	11.70	13.40	15.00
Total length of thread and length of taper at top	0.374	0.568	0.574	0.747	0.759	0.943	0.967	0.984	1.019	1.512	1.575	1.625	1.075	1.781	1.887	2.087	2.300	2.500
Length of perfect thread	0.189	0.290	0.296	0.390	0.402	0.508	0.532	0.549	0.582	0.887	0.950	1.000	1.050	1.156	1.262	1.462	1.675	1.875
Outside diameter of perfect thread	0.405	0.540	0.675	0.840	1.05	1.31	1.66	1.90	2.375	2.875	3.50	4.00	4.50	5.563	6.625	8.625	10.75	12.75
Depth of thread	0.029	0.044	0.044	0.057	0.057	0.069	0.069	0.069	0.069	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100	0.100
Outside diameter of thread at end of pipe	0.393	0.521	0.656	0.815	1.024	1.283	1.626	1.865	2.338	2.819	3.440	3.937	4.434	5.490	6.546	8.533	10.645	12.632
Root diameter of thread at end of pipe	0.333	0.432	0.567	0.701	0.910	1.144	1.487	1.726	2.199	2.619	3.240	3.737	4.234	5.290	6.346	8.333	10.445	12.432
Taper of thread per inch of screw																		
Size of tap drill																		

and genuine wrought iron pipes. It should be remembered, however, that "The superiority of galvanized-iron pipes (also galvanized-steel pipes) disappears when the protective coating (of zinc—the galvanizing) is destroyed." "An examination of the physical appearance of the galvanized pipes tested in this study indicates that the coating had deteriorated with service."

Because copper and brass, as pipe or tubing, does not rust or become rust clogged, but on the contrary maintains its full and original effective area, it is possible to use a smaller size than would be required where corrodible piping is to be installed. Corresponding sizes are given in table 2. As an example, suppose after all calculations a $1\frac{1}{2}$-inch standard size pipe is required to supply the necessary quantity of hot water, this size, when made of copper or brass, should be installed. If, however, you wish to install galvanized pipe, you should use a $2\frac{1}{2}$-inch standard size pipe of this material in order to allow for rusting and rust clogging. If you install a $1\frac{1}{2}$-inch galvanized pipe you will soon find that it is unable to deliver the requisite quantity of water.

The summaries and statements taken from the Fair, Whipple and Hsiao paper do not by any means do justice to this very excellent piece of research. To those interested in this subject a complete copy of the paper should be of great value. The Copper and Brass Research Association has a limited number of copies of this paper and will be glad to send one to you on request.

So much for the experiments on the effects of corrosion. On the more practical side attention is invited to a comprehensive survey undertaken years ago on pipe installations in apartment houses in the city of Pittsburgh. This city has water that is very severe on piping. This survey covered 128 apartment houses located in Pittsburgh. It shows that at the end of four years trouble started on installations with steel pipe and that at the end of eleven years every installation (28 in number) of steel pipe had failed. In the case of 66 installations of genuine wrought iron pipe, failures started at about the eighth year and at the end of eighteen years the 66 genuine wrought iron pipe installations had failed except one. With brass pipe, the bulletin states that at the end of the eighteen years no single failure of the 34 brass pipe installations had been recorded. The results of this survey are equally valid for copper pipe or tubing, where it is used in place of brass.

ECONOMY OF USING COPPER OR BRASS PIPE OR TUBING

The total cost of any installation is made up of material, labor and fixed charges. This total is the initial cost, but by no means always the ultimate total cost. It may be necessary to make replacements within a relatively short time and then the job has to be done all over again at usually a much higher cost, with the inconvenience and losses incident to such shut downs.

The published price of various kinds of piping materials is about as follows:

	Standard pipe sizes *cents per foot*
1 inch galvanized steel	0.08
1 inch genuine wrought iron	0.12
1 inch galvanized genuine wrought iron	0.14½
1 inch brass	0.37
1 inch copper	0.42
1 inch lead (1A—4 pounds per foot)	0.32
1 inch copper water tubing	0.25

Inasmuch as corrodible pipe materials soon become badly coated with rust and rust clogged, it is necessary to use a larger size of pipe. Equating the prices accordingly, we have

	Standard pipe sizes *cents per foot*
1¼ inch galvanized steel	0.11
1¼ inch genuine wrought iron	0.16
1¼ inch galvanized genuine wrought iron	0.19½
1 inch brass	0.37
1 inch copper	0.42
1 inch lead (1A—4 pounds per foot)	0.32
1 inch copper water tubing	0.25

Bearing these prices in mind as compared to the total cost of a certain job, it will be realized at once that the cost of the pipe is only a small part of the cost of the entire installation. It should also be noted that, while copper and brass pipe and tubing cost more than the corrodible materials and, with the exception of copper tubing, more than lead, this difference in an installation would affect the total cost by a relatively small percentage. However, if a proper installation of copper and brass is made the necessity for replacement is eliminated and the slightly higher cost for these materials is more than offset many times by the elimination of expenses necessary for complete replacement.

COPPER AND HEALTH

The effect of copper on health is a subject of such tremendous importance to all users of copper and brass that the conclusions in an article read before the last convention of The New England Water Works Association[4] are given below:

1. The copper content of domestic drinking water after passing through copper or brass pipe or tubing, even if a slight amount of copper is taken into solution, is not harmful to health. It may indeed be beneficial.

2. All water supplies intended for domestic use after passing through a proper installation of copper or brass pipe or tubing will contain but a small percentage of the permissible copper content, if any at all.

3. The permissible copper content of domestic water based on a consumption of one gallon of water a day should apparently not be greater than 20 p.p.m.

4. It is unlikely that health will be injured by copper because, where the copper content is above 5 p.p.m., the taste of the water will generally be so disagreeable that it will be virtually impossible to drink it.

5. No anxiety need be felt about a proper installation of copper or brass pipe or tubing, because it has been shown that with acid water having a pH value of about 6.5, the copper content is generally only about 0.5 p.p.m., even after standing in the pipe lines for an appreciable length of time.

6. The treatment of reservoirs with copper sulphate to rid them of undesirable microscopic growths has no injurious effect on health.

Additional references relating to this subject follow:

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[4] Jour. N. E. W. W. A., 44: 4, December, 1930.

HART, E. B. (also DRS. STEENBOCK AND ELVEHJEM AND WADDELL): Science Supplement, April, 1928.
POLSON: British Journal of Experimental Pathology, 10, No. 4, London.
PARKER AND NYE: Journal of Medical Research, 1921, Vol. 42, p. 461.
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BURSTING PRESSURES

A recognized formula (known as Barlow's formula) that may be used with safety for obtaining approximately correct bursting pressures of pipes and tubes is shown graphically in figure 1.

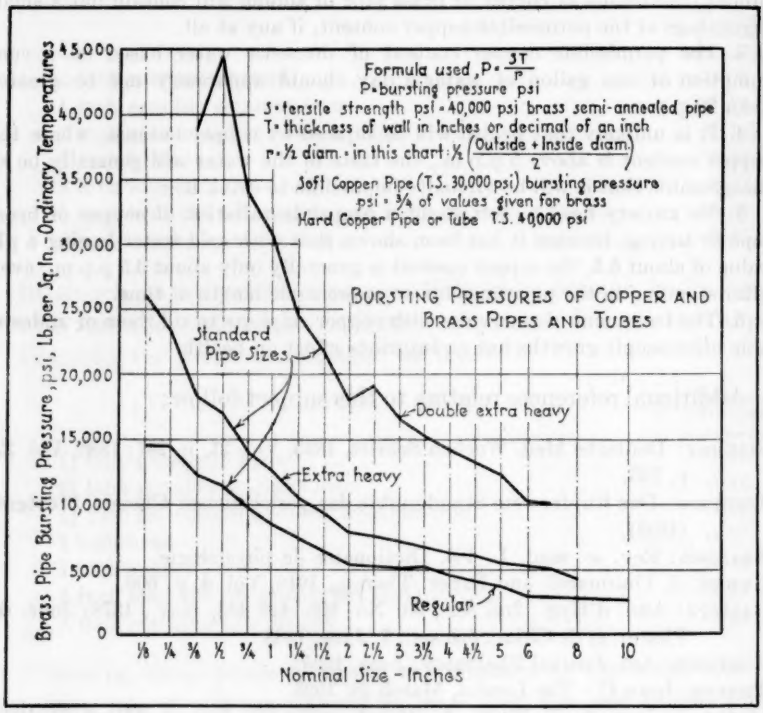
Formula used, $P = \frac{3T}{r}$
P = bursting pressure psi
3 = tensile strength psi = 40,000 psi brass semi-annealed pipe
T = thickness of wall in inches or decimal of an inch
r = ½ diam. = in this chart: ½ (Outside + Inside diam.)
Soft Copper Pipe (T.S. 30,000 psi) bursting pressure psi = ¾ of values given for brass
Hard Copper Pipe or Tube T.S. 40,000 psi
BURSTING PRESSURES OF COPPER AND BRASS PIPES AND TUBES
Standard Pipe Sizes
Double extra heavy
Extra heavy
Regular
Brass Pipe Bursting Pressure, psi, Lb per Sq. In. - Ordinary Temperatures
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
⅛ ¼ ⅜ ½ ¾ 1 1¼ 1½ 2 2½ 3 3½ 4 4½ 5 6 8 10
Nominal Size - Inches

FIG. 1

The formula may be used for practical conditions where the atmosphere is the external pressure or where the external pressure is moderately greater. In this case the difference between the inside and outside pressure represents the bursting pressure being exerted.

It is assumed that gauge pressures are referred to and also that the length of the tube is relatively greater than the diameter.

Barlow's formula makes use of the outside diameter of the tube or one half of this for the radius. Results obtained by using such figures, it is felt, err too much on the side of safety, and, therefore, the chart has been calculated using as the radius one-half of the outside plus inside diameter divided by two or, in other words, the average diameter. A comparison between results obtained by using the Barlow's formula and the one used for obtaining the curves gives the following values for the bursting pressures:

PIPE SIZE	BARLOW—USING OUTSIDE DIAMETER	MODIFIED—USING AVERAGE DIAMETER
inches		
½	10,230	11,700
1	7,700	8,530
3	5,000	5,180
6	3,020	3,120

In other words, the difference on the larger sizes is negligible and, as a matter of fact, also on the smaller sizes, when the safety factor applied is high, as it naturally would be due to actual high pressures. The safety factor will naturally vary with each installation and may be from as low as 2 to as high as 10 or even more.

SPECIFICATIONS FOR COPPER AND BRASS PIPES AND TUBES

Specifications relating to these materials have been issued and may be obtained from the American Society for Testing Materials, 1315 Spruce Street, Philadelphia, Pa.

General data

Brass and copper pipe:

Pressure. Pipe up to 4-inch is tested with a water pressure of at least 1,000 pounds per square inch. Pipe from 4- to 10-inch inclusive with a pressure of at least 500 pounds per square inch.

Lengths. Pipe, is furnished in exact 12- or 16-foot lengths or mill lengths. Exact lengths will not be more than ⅛ inch longer or ⅛ inch shorter than specified.

Temper. All brass pipe will be furnished semi-annealed. Copper pipe is furnished hard drawn. If either grade is wanted for bending this should be stipulated in the order.

Scale. All pipe will be scale free.

Copper tube:

This tube is for use with compression fittings. Such fittings are machined and compress the flared end of the tube. Also suitable for

special fittings where tube is fastened by soldering, brazing, welding, etc.

Lengths. Straight lengths of copper tube obtainable up to 35 feet and in coils up to 60 feet in length.

Pressure. All sizes of copper tube listed have a bursting pressure in excess of 2,000 pounds per square inch.

Temper. All copper tube listed for use with compression fittings will bend readily.

Scale. All tube is scale free.

Cold drawn copper tubing or pipe develops a tensile strength of about 40,000 pounds per square inch, with an elongation of about 10 percent in 4 inches.

Soft annealed copper tubing or pipe develops a tensile strength of about 30,000 pounds per square inch, with an elongation of about 25 percent in 4 inches.

Copper water tubing in comparison with the same lengths of standard size pipe delivers the respective quantities of water shown below:

SIZE		LENGTH	TUBING FLOW
Tubing	Standard pipe		
inches	*inches*	*feet*	
½	½	20	0.7 to 1 per cent less than standard pipe
¾	¾	20	2.2 per cent more than standard pipe
1	1	10–100	5 to 15 per cent more than standard pipe

RESISTANCE TO BURSTING DUE TO FREEZING

The following materials were tested by allowing water to freeze in them in order to determine their relative resistance to bursting due to this cause.

Copper water tubing
Double extra lead pipe
Standard galvanized pipe

The water-filled copper service tube after 12 hours at a temperature below zero showed an increase in diameter of about 12 percent. After thawing the pipe and again filling with water and holding at below zero for 3 hours a further increase in diameter was noted of about 5 percent. A third test showed a further increase in diameter of about 2 percent.

The double extra lead pipe increased slightly in diameter after the first freezing but became very flaky showing a serious disintegration.

Standard galvanized iron pipe burst during the first freezing.

After three successive freezings the copper pipe still withstood freezing strains that caused the other materials to fail on the first freezing.

In a comparative test of bursting pressures sections of copper service pipe, standard iron pipe and lead pipe were filled with water. One end of the pipe was closed with a plug and a plunger inserted in the other. Pressure on the plunger was exerted by a regular testing machine until the specimens burst with the following results:

Bursting pressures

	Bursting pressure pounds per sq. inch
Copper service pipe:	
½-inch	6,120
¾-inch	4,815
1-inch	3,890
Standard iron pipe:	
½-inch	1,140
¾-inch	1,000
1-inch	920

	Extra strong pounds per sq. inch	Double extra strong pounds per sq. inch
Lead pipe:		
½-inch	1,700	2,600
¾-inch	1,500	1,600
1-inch	1,100	1,200

No leaks at joints were noticed with copper tubing and compression fittings.

REFERENCES

For those interested in the installation, bending, inspection, etc. of copper and brass pipes or tubes the following books or pamphlets are suggested, i.e.:

Practical Brass Pipe Plumbing. Obtainable free of charge from the Copper and Brass Research Association, New York.

Some Approved Methods of Testing Plumbing Installations. Obtainable free of charge from the Copper and Brass Research Association, New York.

Pipe and Tube Bending and Joining, by Marks. Published by Isaac Pitman and Sons, 2 W. 45th St., New York. Price, $1.75.

Copper and Health. Obtainable from the Copper and Brass Research Association, New York, without charge.

Water Services That Serve. Obtainable from the Copper and Brass Research Association, New York, without charge.

Copper and Brass Water Services, reprint from Vol. XLIII, No. 2, of the Journal of the New England Water Works Association. Apply to the Copper and Brass Research Association, New York.

Copper and Health, reprint from Vol. XLIV, No. 4, of the Journal of the New England Waters Works Association. Apply to Copper and Brass Research Association, New York.

Hydraulic Service Characteristics of Small Metallic Pipes, reprint from Vol. XLIV, No. 4, of the Journal of the New England Water Works Association. Apply Copper and Brass Research Association, New York.

DISCUSSION

MR. BUCHANAN (London): We, in common with other water works commissions, are interested chiefly in the use of copper pipe for services. Previous to the use of copper pipe, we had been using lead pipe for a period of sixty years, and we found it eminently satisfactory in every way. About ten years ago when the price of copper pipe came down it came into competition with lead, chiefly on account of the price.

I have been rather disappointed in the papers that I have heard in relation to the comparison of copper and lead. I am still of the opinion that there is no advantage in using copper instead of lead, other than that of price. I have not as yet seen any comparison as to the carrying capacity or friction loss in copper as compared with lead, and that is the only advantage that may be in favor of copper pipe.

I would like to have Mr. Schneider's opinion in regard to that.

MR. SCHNEIDER: The published price of 1-inch lead pipe, 4 pounds per foot, is 32 cents per foot, and the 1-inch copper water tubing is worth 25 cents a foot. This does not by any means make up the total cost. The copper tubing may be installed more easily and cheaply than the lead pipe, and, therefore, a considerable saving is effected. As to the carrying capacity, there is no reason why the lead should not convey just as much water as the copper pipe, because after all the carrying capacity of two pipes, other things being equal, depends on frictional resistance. With copper and brass the tendency is for less lime deposit to form than with other materials because

the copper and brass pipe or tubing remains smooth. Where the material rusts or becomes roughened the flow of water cannot so readily carry the lime along. As to friction, I should think with lead, although I do not know it to be a fact, that it is slightly greater.

MR. BUCHANAN: The price seems to depend altogether on market conditions. I am not sure of my data, but I think about 1920, or a little later, copper prices fell rapidly on the market, and lead did not fall so quickly, with the result that from that date until 1927 or 1928 copper pipe for services, including the total cost of installing, was considerably cheaper than lead. For two years following that, copper apparently recovered on the market, and we went back to lead. Then, within the last year those of you who have copper stock know what has happened to copper, and we are now using copper again.

I have been anxious about it because I have some copper stocks myself and I want to encourage the business. Copper is cheaper than lead, but outside of that I do not think there is any particular inducement to use copper in place of lead. I think Mr. Hodkinson would rather install copper pipe than lead because it is easier to handle.

MR. HODKINSON (London): Copper is lighter to handle, and it is easier to make a joint than on a lead pipe, but I think it all depends on the market price of the material as to whether we would use copper or lead. If the price of copper goes up, it will make us consider whether we will use copper or lead. Copper does not take up as much room for storage as lead, and it is easier to haul, and easier to make a joint.

MR. SANDERSON (Toronto): If my memory is correct there are two specifications for copper piping and copper tubes in A. S. T. M. standards. In one the copper content is specified as 99.9 percent and the other specification is slightly under that, and I have been wondering why there are these two specifications. Is it a question of price or manufacture?

MR. SCHNEIDER: The standard has been brought up from 99.880 to 99.900. That is an increase of 200ths of 1 percent in copper content, which is really a negligible factor. In other words, there is

really no reason for making that change as far as I can see, except that it will make a specification for copper pipe a little more rigid, and 99.9 is an easier figure to remember.

There is really no difference as far as the working quality of the pipe is concerned.

MR. SANDERSON: In the recent issue for 1930 there are two specifications.

MR. SCHNEIDER: That is true.

MR. SANDERSON: Why are they there?

MR. SCHNEIDER: I do not see any reason why they should be there. I am a member of the committee and when they revised these specifications they decided to bring the copper content up to 99.9. Probably the reason is that the lower specification, which called for 99.880 copper, has not been revised since 1924 whereas the specification 99.9 was last revised in 1930. If the specification read 99.7 you would have a chance for more impurities and more oxygen. In the case of 99.9, however, you are getting purer and better copper than ever and your tubing specifications should call for this copper content.

MR. HODKINSON: Is there any difference in annealing of the copper. When you are softening copper you heat it to what we call cherry red.

MR. SCHNEIDER: Yes.

MR. HODKINSON: And then you dip it?

MR. SCHNEIDER: Yes. The copper tubing usually furnished by mills has a tensile strength of about 40,000 pounds per square inch. If you wish tubing for bending you should so specify when ordering. You will then obtain a softer tubing which will have a tensile strength of from 32,000 to 34,000 pounds per square inch.

MR. KEITH (Windsor): You state that the copper and brass tubing contains an efficiency of 1.00 when carrying water at a temperature of 140°F. How would these pipes compare if they were carrying

steam? I have found that there is a channeling, or chiseling, when brass pipe is carrying steam.

MR. SCHNEIDER: The corrosion mentioned should not occur in brass pipe. Probably you should use a higher mixture of copper than the one you are using. We are working on the use of copper tubing for steam return lines. Incidentally, steamships are large users of copper pipe for their steam lines.

MR. BUCHANAN: The question is, what is the electrolytic action that takes place when copper pipe is joined to pipes made of some other material, such as galvanized iron, if any?

MR. SCHNEIDER: That depends on the condition of the water. If the water is acid or close to neutral, and most waters are of this character, it will be corrosive. If you put copper and iron together in contact under conditions where the water acts as an electrolyte, electrolytic action will naturally take place at the point of contact and the iron will be corroded. All commercial metals arrange themselves in an electro-chemical series. I choose to arrange them in the order of starting with the most electro-positive and continuing toward the electro-negative end of the series. Zinc, iron, steel, nickel, tin, lead, brass, copper. If you take any one of these metals that is electro-negative to any other one, say for instance, copper, and connect it with iron in the presence of an electrolyte which may be water, electrolysis results and in this instance the iron is corroded. When iron is galvanized with zinc the zinc is electro-positive to the iron, according to the classification above, and acts as a protective coating. However, when the zinc galvanizing has been pierced and the underlying iron is in contact with the zinc in the presence of acidified water, galvanic action results and the zinc very rapidly goes into solution. It is for this reason—because zinc is electro-positive to the iron—that it is a protection to the iron pipe.

MR. BUCHANAN: There would not be any trouble of that kind with alkaline water?

MR. SCHNEIDER: No, there should not be. Where the possibility of electrolysis exists it may be well to insulate against it. Connecting the copper pipe to an iron pipe under such circumstances

involves the use of proper unions. Where larger sizes of pipe are to be joined together it is advisable to use a ferrous cast flange on the iron pipe and a brass flange on the brass pipe or a high copper content brass or bronze flange on the copper pipe. Between the two flanges a sheet of lead should be inserted, the bolt holes should be lined with lead, bronze or brass bolts should be used and lead washers. It is possible in this way to insulate one pipe from another.

MR. MARTINDALE (Sudbury): For a number of years we used lead exclusively for service pipes up to 1 inch in diameter. Our experience with lead pipe has not been entirely satisfactory. Our specifications call for pipe made from virgin pig lead, but I am sorry to say that in some cases we do not get it. We have had a number of failures of lead pipe, and have had to replace the pipe, but these failures in the lead have been due to carelessness in the makeup, and after the lead has been in use 12 to 15 years it splits longitudinally, and brings about numerous cracks. We submitted samples to the manufacturers, and they all promptly disclaimed responsibility. In those days the manufacturer's name was not on the article. For the last 5 years we have been using copper pipe almost exclusively.

Our experience with copper service pipe has been entirely satisfactory, with one or two exceptions. In the first place, there is nothing in the market that will equal copper service pipes for convenience of handling. It is easy to make, and also easy to tighten up the joint, and it can be put in by comparatively inexperienced labor. It takes a man with some experience to make a first class lead pipe joint. We used, for years, a lead flange coupling. Wiped joints are rather difficult to make and we do not always have competent men to wipe the joint, and therefore we have been using a lead flange joint entirely. It is a very satisfactory joint if properly made, but it is harder to make than a copper joint. We like the copper pipe on account of its convenience of handling, ease of bending and getting around corners.

We had an experience with copper service pipe this year which I should like Mr. Schneider to explain. It was a service pipe running into the street railway carbarns, and the copper was decomposed as to the exterior surface of the pipe, so much so that it had to be repaired. I think Mr. Schneider will probably say that was caused by electrolysis, but I am not sure whether it was or not.

Close to this pipe, the ground was filled with cinders, and I am not

sure whether this damage was caused by the railway cinders, or by a direct electric current.

I had another experience with copper pipe this winter. We had conduits laid down for electric cable, and these conduits became filled with water and the water froze. I had to devise some means of getting the water out of these conduits, which were made of fibre laid in concrete, and I used $\frac{5}{8}$-inch copper tubing in 60-foot lengths to carry steam into the conduits. I was quite surprised when the material came back to the store room, to find that the copper tubing, due to the use of the steam and its immersion in the ice cold water, was hard and brittle, and we could not possibly use it. We tried annealing it by reheating it and allowing it to cool gradually, and we were successful in salvaging some of it, but most of it had to be scrapped.

MR. SCHNEIDER: I can answer the first part of your question easily, but I am not so sure about the other point raised.

All cinders form sulphurous and sulphuric acid and where pipe is buried in such cinders corrosion is certain to occur. Therefore, you should never under any circumstances put copper or brass, galvanized iron or steel or even lead in a cinder fill unless the pipe is protected against the leaching from such cinders. I do not say that was the entire cause of your trouble. I do feel, however, that the chemical action from the cinders had a lot to do with it. If you have a current in your pipe line, there is only one thing to do and that is to get rid of it by proper grounding or in some other way.

MR. BUCHANAN: It is very possible that it was a combination of the two. Cinders would make a good contact for your electricity.

MR. SCHNEIDER: If it was a combination of the two, the pipe would go very rapidly.

With regard to the pipe being used to thaw out conduits, it seems to me that the two temperatures should neutralize themselves, and the copper piping should have been relatively at a uniform temperature. When the steam is turned on it is possible that the copper might have been hot, and it might be made soft, but not brittle. Is it possible that the tubing was inserted in the conduit and that the water froze in it?

MR. MARTINDALE: The water just flowed out.

MR. SCHNEIDER: I shall be very happy to have you submit some of the tubing in question to us. We shall be glad to send it to the laboratory for chemical and microscopical analysis and report fully to you.

MR. HODKINSON: We have some pipe alongside of a gas works, and we use lead pipe, and it corroded, and we put in copper and it lasted longer.

CENTURY OLD CAST IRON WATER MAIN

By Seth M. Van Loan[1]

On the 29th of April, 1931, in Philadelphia, Mayor Mackey, Director Murdoch of Public Works and Chief Hayes, Bureau of Water, participated in the observation and examination of a section of 6-inch cast iron water pipe which was laid and had continued in active service in the distribution system since the year 1831.

The inspection was made an occasion for a radio program which was broadcast from a stand erected near the excavation at 5th and Federal streets.

The Mayor, the Director of Public Works and the Chief of the Bureau of Water delivered addresses before several thousand and amplifiers projected the speeches over the air through a public address system installed for the occasion.

Examination of the pipe disclosed it to be in good condition and capable of functioning for an indefinite period.

This pipe was part of the system laid subsequent to the year 1817, before which date practically all the water mains laid from 1800 in Philadelphia and other cities in the United States were of wood.

The piece of pipe removed was an importation from England and is the bell and spigot type made in 9-foot lengths. A curious feature of these pipes, which was mentioned in a paper before the American Society of Civil Engineers in 1914 by Marshall N. Pugh, is that each length has three or four rings about it, approximately $1\frac{1}{2}$ inches wide, which were presumably to afford places with heavier walls in which to tap the pipe. Although these original pipes are now only a portion of Philadelphia's present great system, nevertheless they still constitute a considerable mileage.

Mayor Mackey and the other officials, who superintended the uncovering of the pipe, took the occasion to announce to the citizens of Philadelphia, the City Administration's plans for developing a larger and more complete water system for a section of the city which had formerly been devoted to private homes but which is now becoming

[1] Deputy Chief, Bureau of Water, Philadelphia, Pa.

the location for large apartment houses. This section lies between Broad street and the Schuylkill River and south of Market street.

In 1817, Philadelphia had completed a sixteen-year study of devices for carrying water and from such date practically all the new work was of cast iron character which as time passed replaced the old wooden mains. The records of these studies are still available and make an interesting exhibit. Cast iron had not been used in this country for the purpose up to that time. In Europe, however, there had been some noteworthy installations, the earliest being that of the parks of Versailles, where in 1664, by order of Louis XIV, cast iron pipes were laid from the reservoirs of Picardie to those of Montbaron, together with a spring water conduit, to supply the town as well as the fountains. These cast iron pipes, now 267 years old, are still in service.[2]

Previous to the use of cast iron pipe and even during the early days of its adoption, pipe made of wood and some made of lead were used. In the City of London much wood pipe was used, but old reports from London state that wood pipe had to be completely renewed every 20 years. The first log pipe laid in Philadelphia was in 1799. It had an outside diameter of 18 inches and a bore of 6 inches. Specimens of this may now be seen in the Franklin Institute.

[2] This JOURNAL, May, 1931.

SOME PROBLEMS OF WATER WELL CAPACITIES[1]

By Howard O. Williams[2]

Water wells may be classified in two general groups, namely; (*a*) wells which obtain their supply of water from hard rock formations, and (*b*) wells which obtain their water supply from unconsolidated material such as sand and gravel.

The problems which are involved in a consideration of the capacity of wells ending in hard rock, and the mechanics of increasing their capacity, are much less clearly defined and much less complex than for sand and gravel wells. For this reason the following discussion will be confined chiefly to wells ending in sand and gravel, and only the most important considerations in conjunction with wells ending in hard rock formations will be mentioned.

Frequently, rock wells involve greater problems in the mechanics of drilling than do sand wells; likewise, the problem of sanitation demands more skill on the part of the rock well driller than the sand well driller. In the matter of yield, however, the possibility of obtaining increased capacity by the proper application of developing methods, based upon a thorough understanding of the characteristics of sand and gravel wells, is very much greater than in the case of wells finished in hard rock, and it involves the use of much greater skill on the part of the operator.

ROCK WELLS

Wells that are finished in hard rock formations may be divided into two general classes, namely:

1. Those which obtain their water supply from consolidated formations such as sandstone that contain water chiefly in the original interstices of the material.

2. Those which obtain their water supply from formations that contain water chiefly in secondary interstices such as joints, fissures, crevices, or solution channels.

[1] Presented before the Indiana Section meeting, February 27, 1931.

[2] Office Engineer, Edward E. Johnson, Inc., St. Paul, Minn.

Wells finished in sandstone yield water by virtue of the interstices between the grains of sand and the cementing material which form the stone. In the harder sandstone, water may also be contained in secondary interstices; but since practically all investigations and formulae are based on uniformity of structure, we shall deal with non-fractured sandstone only, in this paper.

Slichter and King (1) made extensive and monumental investigations for the United States Geological Survey over thirty years ago dealing largely with sandstone wells, and as far as the writer has found, nothing better has been written since that time on the hydraulics of wells of this type. The data which they presented were largely of a theoretical nature, which fact should be considered in any attempt to apply them to field problems. Indeed, it may be said that their findings are not of real positive value in determining the yield which might be expected from sandstone wells; but all of the data have a very definite value of a comparative nature. To explain this a little better, it may be stated that, if the general geological conditions and the nature of the sandstone were known in a certain location, it would not be possible to apply their findings to determine the yield of a well at that location with any expectation of accuracy. It would be possible, however, to determine with fair accuracy from the data which they give what yield might be expected from additional adjacent wells if the yield of one well were already known; or if one well of large diameter were sunk down to the sandstone formation, it could be determined with reasonable accuracy how much the yield could be increased by drilling inside of the large well one or more smaller bore holes a given distance into the formation.

For our purposes, however, we may deal with this type of well in a more general fashion and make certain general recommendations for procedure.

In the first place, except under unusual conditions, it is poor economy to stop a sandstone well as soon as, or shortly after, the formation is encountered. The capacity of the well can usually be very materially increased at small cost by continuing the bore through the entire thickness of the formation. In a formation of uniform structure, the increase in yield will be greatest for the first portion of vein which is penetrated, and will become less and less as the bore is extended deeper. It is, however, usually profitable to continue the bore any distance up to 400 feet into the formation.

Under certain conditions, it is possible to increase with safety the

yield of a hard, tight sandstone by blasting and then drilling and cleaning out the bore. This produces a reservoir-like enlargement of the bore with fractures extending outward into the vein. Good judgment is required to determine when a sandstone well should be "shot" and a skillful operator should do the work. Carelessness in blasting has ruined many wells which might otherwise have been successful.

The yield of wells that end in sandstone is sometimes limited because the formation is not tightly cemented and hence sand issues with the water whenever the well is heavily pumped. Such a condition is hard to remedy. An experienced driller can sometimes overcome the trouble by "shooting" the well with dynamite, cleaning it out, and then packing it with clean, fine gravel. Occasionally, trouble from sand pumping can also be successfully overcome by placing in the stratum a well screen about half the diameter of the bore and packing fine gravel around the screen.

The second class or rock wells includes those which obtain their entire water supply from secondary interstices; Meinzer (2) lists three types of secondary interstices, namely:

1. Joints and other fracture openings.
2. Solution openings.
3. Openings of minor importance, such as those due to mechanical erosion, recrystallization, etc.

The first two are the only types of significance to us.

Wells finished in limestone, dolomite, and other rocks of calcareous nature are subject to solution in water, and it is common to find interconnected solution passages in these rocks which range in size from tiny capillary openings to huge caves such as the Carlsbad Cavern of New Mexico. Naturally, when these solution openings have contact with an extensive infiltration area, they are capable of carrying immense quantities of water to any wells which may pass through these channels. Many wells that end in limestone or dolomite are abundant yielders of water; but unfortunately the water is frequently very high in total solids. Another serious objection to all rock wells that derive their supply from formations that contain water in secondary interstitial openings is the fact that they are more likely to contain polluted water. It has been estimated that approximately 25 per cent of all the rock wells in the state of Wisconsin are polluted to some degree, chiefly due to the extensive transmission of polluted waters through the limestone which overlies the sandstone which is the most common source of water supply from rock wells in the state. Similar conditions probably obtain in other states.

Most rock wells of this general type present a greater problem in sanitation than in yield. There is little real problem of yield. It is chiefly a matter of chance whether the bore of the hole happens to connect with good joints, bedding planes, or solution channels. There being no formula for chance, we may dismiss the matter of yield from such wells by stating the following general recommendations:

1. The borehole in such wells should be continued until ample water is obtained or until the entire formation is penetrated, in the case of limestone, dolomite, etc.; and in such rocks as granite, quartzite, and others depending upon joints, crevices, and bedding planes, the drilling should be continued as long as there is evidence of any further openings, and discontinued when any considerable thickness has been drilled without signs of large interstices. This is one place where the skill and judgment of the driller is worth more than any formula ever compiled.

2. Where there is water obtained, but not in sufficient quantities, the yield is frequently increased by blasting. The explosion may set up new joints which will extend to adjacent water-bearing joints or channels not otherwise connected with the borehole. Blasting should not be used on all rock wells because there is almost as much chance of ruining a good well as to improve a poor well by blasting. Only a skilled driller should attempt to do this work.

SAND AND GRAVEL WELLS

When we enter into a consideration of the yield of sand and gravel wells, we are confronted with a problem of growing importance due to the increased demand for wells of this type. This demand is largely the result of modern well finishing methods which make the sand and gravel well very generally superior to the rock well in combined quantity, quality, sanitation of supply, and overall gallon-year cost. We also find a problem in which results depend largely upon the mechanics of development. When we deal with sand and gravel wells, we reduce the factor of chance to a minimum and substitute in its place the factor of development which must depend for its successful application upon a proper understanding of certain fundamental laws of hydraulics and mechanics.

This is a problem of such magnitude that it cannot be covered in any detail in this brief paper, but in order to give an idea of some fundamentals, it should be profitable to touch upon certain outstanding points.

A great deal of research into the movement of water through sand has been done in the past fifty years by such eminently able engineers and geologists as King, Slichter, Hazen, Norton, Meinzer, Stearns, and others. In recent years, the German engineers have evidenced much interest in the subject. The results of many valuable experiments have been recorded and numerous formulae developed. Doubtless, these formulae have practical value in studies of filters, water prospecting to determine probable maximum capacity of the water supply in a given territory, and similar applications. However, when an attempt is made to apply these formulae to individual wells in sand and gravel the results are worse than unsatisfactory—they are, in fact, misleading.

As an example, the following formula (3) to determine the expected yield of a well or the proper diameter of well for a required yield is advanced in one of the best of recent volumes on the general subject of water supply. The authors refer to the German engineer, König, as their source:

$$D = \frac{Q\ Cp}{H\ V\pi}$$

in which D = diameter in feet.
Q = quantity in c.f.s.
Cp = coefficient of permeability. (Apparently equivalent to U. S. G. S. coefficient of permeability expressed in hundreds instead of units.)
H = height of screened section of vein.
V = velocity in feet per second.

In solving, they assume as a normal example,

$$Cp = 4,\ V = 0.0033$$

Applying this with 10 feet of screened vein and $Q = 0.5$ (225 g.p.m.), we have:

$$D = \frac{0.5 \times 4}{10 \times 0.0033 \times 3.14} = 19.3$$

which would indicate a well about 20 feet in diameter to yield 225 g.p.m.

According to this formula as applied, a 12-inch well with 10 feet of vein screened should be expected to deliver only 12 g.p.m.

Such results are out of all proportion, of course, and to attempt to use such a formula successfully in the field is out of the question.

If we examine these different formulae carefully, we find that they are all based on (*a*) a very low and constant velocity of the water, (*b*) a constant effective grain size, coefficient of uniformity, and coefficient of permeability for all points in the vein around the well, and (*c*) no disturbance of the sand as originally found in the vein. When we realize that modern methods of sand well finishing and development are based on the use of far higher velocities than are commonly applied in these formulae, that there is a definite purpose to produce a change in effective grain size and the coefficients of uniformity and permeability, and that strenuous efforts are used to disturb the sand around the well end, it is very easy to see why these formulae fail so completely in practical application.

Some day, perhaps, we shall have formulae which we can apply with better results to individual wells; but a close study of the subject for several years leads me to state that the observation of the following axioms is of far greater value at present.

1. Before drilling a well to furnish any considerable quantity of water, in an untried territory, first determine the most favorable locations from the standpoints of geology, sanitation, and convenience, and then sink one or more test wells. A skilled observer can determine more from a good test well than from all the formulae in existence.

2. Determine the type and size of permanent well from the findings of the test work, a careful analysis of the sand or gravel as found in the vein, the mechanical convenience in sinking and developing, and pumping efficiency.

3. Especial emphasis should be laid upon development of the well to its fullest capacity by the use of the most efficient screening device and proper application of developing methods. This is particularly important because capacity, drawdown, and permanence are functions of intake area and development; and overall gallon-year costs are, in turn, functions of capacity, drawdown, and permanence.

We can best demonstrate these axioms by briefly touching on the hydraulics of sand wells and studying their behavior under different conditions.

HYDRAULICS OF SAND WELLS

Figure 1 illustrates what may be termed the "normal hydraulic gradient," during pumping, of a well finished in sand or gravel formation before any development has been done. By "hydraulic gradient" is meant the slope of the surface of the water in the vein.

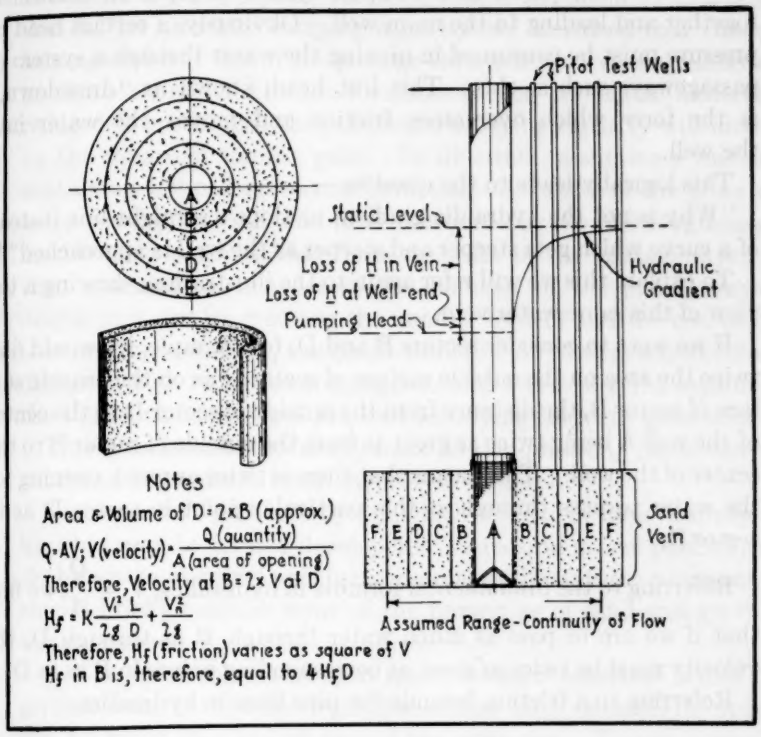
Pitot Test Wells
Static Level
Loss of H in Vein
Loss of H at Well-end
Pumping Head
Hydraulic Gradient
A
B
C
D
E
Notes
Area & Volume of D = 2 × B (approx.)
Q = AV; V (velocity) = Q (quantity) / A (area of opening)
Therefore, Velocity at B = 2 × V at D
$H_f = K \frac{V_1^2}{2g} \frac{L}{D} + \frac{V_2^2}{2g}$
Therefore, H_f (friction) varies as square of V
H_f in B is, therefore, equal to 4 H_f D
F E D C B A B C D E F
Sand Vein
Assumed Range - Continuity of Flow

FIG. 1. ILLUSTRATION OF THE HYPOTHESIS OF THE NORMAL HYDRAULIC GRADIENT OR SLOPE IN A WATER WELL PRIOR TO DEVELOPMENT

It will be noticed that a main well is represented having a number of small pilot wells drilled down to the water formation in a row away from it. If we could measure the water level in the main well when pumping at a given rate and before any development whatsoever has taken place, and measure the levels of the water in the pilot wells at the same time, we would find that the water levels would keep getting lower and lower as the main well is approached, and if the points were connected together by a smooth curve it would appear as shown in

the illustration. It would be called the hydraulic gradient of the well at the given rate of pumping.

The first question which might be asked is, "What causes drawdown?" In my opinion, the best way to approach this question is by thinking of the small voids or passageways in a sand vein through which the water passes as a complex system of pipes all connected together and leading to the main well. Obviously, a certain head or pressure must be consumed in moving the water through a system of passageways such as this. This lost head, known as "drawdown," is the force which overcomes friction and moves the water into the well.

This logically leads to the question—

"Why is not the hydraulic gradient or slope a straight line instead of a curve which gets steeper and steeper as the well is approached?"

To explain this we will refer again to the illustration, showing a top view of this same water vein.

If we were to consider sectors B and D, for example, we would find twice the area on the outside surface of sector D as on the outside surface of sector B, the distance from the outside of sector D to the center of the well A being twice as great as from the outside of sector B to the center of the well. This means that there is twice as much opening for the water to pass through at this particular point in sector D as in sector B.

Referring to the fundamental formula in hydraulics, $V = \frac{Q}{A}$, we find that if we are to pass as much water through B as through D, the velocity must be twice as great at corresponding points in B as in D.

Referring to a friction formula for pipe lines in hydraulics,

$$H_f = K_1 \frac{L_1}{d_1} \frac{V_1^2}{2g} + \ldots . K_{n-1} \frac{L_{n-1}}{d_{n-1}} \frac{V_{n-1}^2}{2g} + \frac{V_n^2}{2g}$$

we find that the friction loss, or the pressure head loss due to friction, varies as the square of the velocity, regardless of the complexity of the system, the friction loss becoming four times as great if the velocity is doubled, nine times as great if the velocity is tripled, etc. Thus, we see that the loss in pressure head must increase approximately inversely as the square of the distance from the well—if conditions were uniform and stable, it would be in exact proportion.

The gradient curve in an undeveloped well, therefore, assumes this

shape or form. It may be less in total vertical depth, but it is bound to have this same general form. This brings us to the next question.

"How can this drop in the curve be reduced? How can its form be changed so that it will approach a straight line?"

To answer this, let us think again of these passageways in the vein as a series of small pipes. Knowing that if the diameter of a pipe is doubled its area and carrying capacity are increased four times, it becomes evident that if the passageways in a sand vein are increased in diameter, and if the total volume of non-permeable material is reduced to accomplish this, the water-carrying capacity will increase by the square of the net gain. To illustrate, assuming a unit area containing 100 passageways of unit size, one-fourth of the gross area being openings or voids and three-fourths being the enclosing non-permeable sand particles that form the walls of the passageways; and then assuming an increase in the diameter of the passageways to double unit size by reducing the number by half and the non-permeable area by one-third, the result would be 50 passageways of 2-unit diameter, with twice the opening area and water-carrying capacity of the 100 previous passageways, and occupying the same gross area.

This consideration naturally leads to the question, "How can this condition be produced in actual practice in a well?"

There are two general ways in which this can be done; namely, (*a*) by what may be termed development or natural gravel packing, and (*b*) by artificial gravel packing. Before outlining these processes, we should briefly consider some of the properties of sand and gravel as water-bearers.

1. The total volume of open space between spherical bodies in a given volume is the same regardless of how large or small the spheres are, provided they are all graded the same size. To put it another way, there is the same total void in a pailful of sand grains which are all 0.001 inch in diameter as in a like pailful of baseballs.

2. If some finer sand is put in the voids between the larger spheres, there will be a reduction in the total void. This means that fine sand mixed into coarse sand or gravel reduces the porosity or water-containing capacity of the coarse sand or gravel; and it will also reduce its permeability or capacity for yielding water.

3. As the converse of paragraph 2, if large pebbles or gravels are mixed with fine sand, the result will be less void or open space than before, and the water in moving cannot go through the pebbles but must go around them; thus, porosity and permeability are reduced in this way, also.

These first three facts bring us to a realization that this matter of mixing sand and gravel requires considerable thought as to the grading of the material. Uniformity of grading is in most cases of even greater importance than the general or average fineness or coarseness of the material.

4. As a final consideration, it is vital to take cognizance of the matter of "bridging" of sand grains. There may be some coarse and some fine material in a vein, and it might be thought possible to remove all of the fine material within the desired range by pumping vigorously. But this cannot be done because the smaller grains will form bridges, somewhat like an arch, across the opening between the larger grains and refuse to be moved as long as the flow of water continues in one direction. That is one of the main reasons why straight pumping is seldom a sufficient means of development for wells in sand and gravel.

THE DEVELOPED OR NATURAL GRAVEL-PACK WELL IN A VEIN OF MIXED COARSE AND FINE SAND OR GRAVEL

A screen of the proper construction is inserted in the vein at the bottom of the well and sealed to the well casing. A single line of well casing is used and no provision is made for the insertion of gravel artificially. Pumping of the vein and some method of surging and development are provided. The easiest way to think of what happens in the bottom of the well is to think of the sand vein as being made up of a large number of minute sectors of sand something like the sectors A, B, C, etc., shown in figure 1. As the finer material is pulled through the slots of the screen, all but the coarsest material next to the screen is removed. Then the finer material in the second sector moves to the first sector, that from the third moves to the second, from the fourth to the third sector, etc. As this is continued, the sand grains gradually arrange themselves with the largest and most uniform only next to the screen; the largest and next smaller size, in the second sector; the largest and next two smaller sizes, in the third sector, etc., with the mixture being more uniform and more permeable as the screen is approached, and—a thing which is vitally important—with *each sector graded in exact relation to the one next to it so that each acts as a stabilizer for the one further on.* Regardless how hard the pumping may be applied, this relationship will not be disturbed even though the nature of each sector may be changed.

ARTIFICIAL GRAVEL PACKED WELL IN VEINS WITH MIXTURE OF COARSE AND FINE SAND OR GRAVEL

A large outer casing is sunk to or through the sand vein, and a small casing and screen is set inside the outer casing. A coarse aggregate is inserted directly around the screen. The vein is assumed to be composed of a mixture of coarse and fine material. As soon as pumping is started, finer material pulls into the coarse material; and since there is a mixture of coarse and fine grains in the vein, the coarse will be held back and the fine will be pulled into the packing gravel, and the well then becomes a job for natural development just as if no gravel had been added artificially. The only advantage is the possibility of slightly larger gravel directly around the screen, which probably will add a little to the permeability, although it will not add enough to pay for the extra cost of this type of construction, except in very shallow wells under ideal conditions.

ARTIFICIAL GRAVEL PACKED WELL IN VEINS OF FINE SAND

The procedure is the same as for veins with mixture of fine and coarse material. If too coarse packing gravel is used in such a case, the fine material will enter the coarse material, filling up the voids between the coarse grains and reducing the permeability, thereby increasing the velocity or speed of the water. This, in turn, will cause more fine material to be pulled in, causing still further reduction in permeability, which will again cause an increase in velocity, and a vicious circle is started which results either in clogging or sand pumping. The aggregate being too coarse and not extensive enough, it cannot build up the graduated wall of assorted sand and gravel around the screen. The only remedy is to replace it with finer aggregate, or to increase its range. This is usually a difficult and expensive task, and it is one of the reasons why it is never safe to pull either inner or outer casing out of an artificial gravel-packed well. When such a gravel-packed well is used, it should always be required that both an inner and outer casing be used and both be left permanently in the well.

COMPARISON OF TYPES

Summarizing, we find that the ordinary artificial gravel-wall or gravel-packed well in a vein of mixed coarse and fine material must be packed with material similar to the coarse material in the vein if the

well is to be pumped heavily, or the gravel pack must be made very extensive. When it has an ordinary pack of properly graded gravel, it promptly turns itself into a natural packing job, which requires the same features in a screen device as the ordinary natural developed well in order to be fully successful. When artificially packing a well in fine sand, it is very essential to make the packing extensive and to hold down the rate of pumping so that it does not start the fine sand moving into the packing gravel. Some such wells are spoiled in testing.

The fundamental considerations necessary to determine what method to follow are:

a. Ordinary fine sand with reasonable permeability and good uniformity is usually best screened without packing and with a relatively fine slot screen rather than the coarse slot screens such as are used in development.

b. Very fine and uniform sand is best artificially gravel-packed.

c. A mixture of coarse and fine sand is best finished with naturally developed pack, using as coarse a slot of screen as an accurate analysis of the material shows to be advisable.

Each type of well has its place. Unfortunately, indiscriminate propaganda has spread the opinion in many quarters that the artificial gravel-wall or gravel-packed well is a panacea for all the ills of sand wells. This is far from true. There are some places where this type of well is superior to others, as outlined above; but in the vast majority of cases, the natural developed or ordinary screened well will prove much superior in gallon-year cost. In fact, there are many cases where artificial gravel-packed wells are decidedly inferior in production to other types of wells that are less costly to install.

In any type of sand well four important features must be specified in the screening device used:

1. Construction with slot rather than square or round opening. This is to prevent plugging, as a grain can make contact on a slot at two points only.

2. The slot must be constructed with sharp outer lip and abruptly widened inner opening. This will prevent any grains which pass the outer lip from wedging and clogging the opening.

3. Maximum area of opening is essential because velocity is an inverse function of intake area, drawdown is direct function of velocity, and pumping cost is a direct function of drawdown. Thus, pumping cost is an inverse function of intake area.

4. Non-corrosiveness. All natural waters are corrosive to varying

degrees, and the permanence of the well will commonly depend in like degree on the non-corrosiveness of the screen.

YIELDS

Before concluding, it will be well to devote a few moments to a consideration of general factors bearing on the matter of the yield of all types of wells.

First: the diameter of the bore.

In flowing wells drilled into sandstone, the influence of diameter of bore is calculated by King (4) as follows:

Assuming the yield of a 2-inch well as 100 percent, the yield of a 6-inch well would be 115.8 percent, and the yield of a 12-inch well would be 126.07 percent.

In pumped wells drilled into sandstone, the maximum yield would follow the same proportion; but the limitations of pumping equipment would be found to be the controlling factor in most cases, except where the yield is weak.

In wells finished in other kinds of rock, the yield will not follow any definite proportion to the diameter, but must depend upon the chance of striking favorable joints or channels.

In wells of sizes large enough to permit adequate development in sand and gravel—roughly 6-inch and larger—my observations in the field have shown that all other factors being equal, the diameter of a well will influence the yield approximately in accordance with the following formula which I have developed for purposes of rough estimation:

$$Y_2 = \frac{(1 + \sqrt{n})}{2} Y_1$$

in which Y_2 = yield of larger well.

$n = \frac{d_2}{d_1}$; d_2 and d_1 being diameters of larger and smaller wells, respectively.

Y_1 = yield of smaller well.

Controlling factors in determining the diameter of a well are:

1. Mechanical requirements of drilling and development. A well of too small diameter in rock presents serious problems, such as insufficient weight of drilling tools, lack of provision for reducing diameter, if necessary, etc. Very large sizes, on the other hand, require

heavy and expensive equipment. Small wells finished in sand and gravel involve the additional disadvantage of being difficult to develop effectively.

2. Efficiency of pumping equipment will establish a minimum size which is commonly larger than the actual requirements of maximum yield.

3. Of less importance, and yet worthy of consideration, is the actual increase in yield which is the net result of increased diameter.

As a generalization, based on a knowledge of a large number of wells in all parts of the country, I feel warranted in recommending for municipal and industrial wells under normal conditions, a minimum diameter of 8 inches and a maximum diameter of 24 inches. Wells smaller than 8-inch diameter seldom prove adequate for municipal or industrial purposes, and wells larger than 24-inch diameter are generally expensive out of proportion to any advantages they may have.

One of the most vital matters to be borne in mind is the arrangement of wells when two or more are sunk into the same stratum. In the first place, if the direction of flow of the ground water is known, the wells should be set in line across the line of flow and not parallel to it. If a river, spring, lake, etc., is nearby, it can usually be assumed that the ground water flows toward the body of water. A study of the underlying geologic structures and their inclinations, together with information obtained from surrounding wells, will ordinarily indicate the direction of ground water movement.

The spacing of group wells is of much importance in order to strike an economical balance between interference in yield and cost of pumping, piping, etc.

Slichter (5) gives the following data based on mathematical calculations:

Given two 6-inch wells pumped at given drawdown, the percentage of interference will be as follows:

Distance apart	Interference
feet	*percent*
5	38.33
10	34.53
100	20.09
200	15.35
400	10.51
1000	6.23

If three wells are placed in line equidistant apart, the following table applies:

Distance apart	Interference
feet	*percent*
5	55
10	50.7
100	30.9

Two wells spaced 10 feet apart may be expected to yield about 31 percent more than one well alone. If a third well were placed between them, the combined yield of the three would be only about 35 percent more than one well.

If the same two wells were spaced 200 feet apart, their combined capacity would be 69 per cent more than one well alone. If a third well were placed between them, the combined yield of the three would be about twice the yield of one well alone.

From these data it would appear that an economical average of spacing between wells would be about 100 feet. A more exact figure could be determined only by test wells and calculation of interference and transmission costs.

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WATER SOFTENING AT BLOOMINGTON, ILLINOIS[1]

By Charles H. Spaulding[2]

The project of providing soft water for the city of Bloomington has several aspects, economic, financial, engineering, etc. This paper will discuss briefly some of the chemical features.

Soft water has been obtained in this case by two steps; first, change from a hard well water supply to comparatively soft surface water; second, softening the new supply with lime. The first step brought the hardness down from 800 in the old supply to 230 p.p.m. in the new, a reduction of 71 percent. The second step reduced the hardness of the new supply to 60 p.p.m., a further reduction of 74 percent. From the rank of one of the hardest municipal supplies of the state, Bloomington has taken two jumps to the place of one of the softest. On the basis of softness alone this supply is equaled by only six or seven smaller cities and villages in the state. When clarity, safety and potability are considered, it is entitled to rank among the best.

THE LAKE

The construction of a lake was doubtless determined in large measure by the demands of quantity as well as quality, but it would have paid the community as a softening project alone. The cost of softening the well supply from 800 to 230 p.p.m. would have been over $150.00 per day for lime and soda ash alone on the present low pumpage of 2.5 m.g.d. The investment is approximately 1.25 million dollars, including lake, purification plant and pipe line.

Previous to the construction of this lake the writer ventured the prediction that such a lake fed by a steam originating in the Bloomington moraine would have about the same hardness as Lake Decatur, which has the same origin and which was reported at that time to contain 200 p.p.m. hardness. This was based on records of the United States Geological Survey[3] showing close similarity in the

[1] Presented before the Illinois Section meeting, April 29, 1931.

[2] Supervising Chemist, Water Department, Bloomington, Ill.

[3] U. S. G. S., Water Supply Paper No. 239.

quality of streams rising in this vicinity. Recent analyses have shown that the waters of the two lakes, Decatur and Bloomington, can scarcely be distinguished on the basis of hardness determination. The available records[3] show an average throughout the year for streams of this origin of 247 p.p.m. This may be compared to 230 p.p.m. in Lake Bloomington during a year of drouth. It appears that we may expect 90 percent of the stream average during dry seasons, and 80 percent during normal years.

An interesting phenomenon has been observed which brings about a reduction in hardness of lake water. It is well known to water chemists and others that chlorophyll-bearing algae produce oxygen in water under the influence of sunlight, the source of the oxygen being carbon dioxide in the water. The plant cells utilize the carbon and half the oxygen in the production of starch; while the other half of the oxygen is set free. I have not seen any reference to this as a softening reaction in the literature, but such it is in fact. The removal of carbon dioxide in this manner has the same effect as its removal by lime, namely precipitation of normal calcium carbonate. Without any rain or runoff, and in the face of a falling reservoir stage, the hardness dropped from 234 in May to 197 p.p.m. in June, and resulted in a saving of $6.00 per day for lime. Thus the much maligned algae have paid dividends. With the return of lower temperatures in the fall, and the decline of the algae population, the hardness again rose and finally exceeded the previous spring records.

THE WATER SOFTENING PLANT

The water softening plant has been admirably designed from the standpoint of flexibility. The two mixing tanks and two clarifiers can be used either in parallel or in series, the intent having been to use series operation during turbid water conditions, at which time the first clarifier would act as turbidity remover, and the second for softening.[4] A minor change in the piping before starting operation made it possible to recarbonate the water before it entered the second mixing tank, and the addition of sludge returning facilities later made possible the full realization of softening with lime, at our present rate of operation.

[4] L. R. Howson, *The American City*, November, 1930.

The sequence of treatment under the present conditions at the rate of 2.5 m.g.d., which is one-half the rated capacity of the plant, is as follows:

	hours
1. Mixing with excess lime	1
2. Settling in 1st clarifier	4
3. Recarbonation	1
4. Mixing with sludge from 2nd clarifier	1
5. Settling in 2nd clarifier	4
6. Secondary recarbonation (optional) at clarifier outlet	
7. Filtration	

Excess lime is added first, sufficient to combine with all magnesium and leave 50 to 60 p.p.m. caustic alkalinity. Caustic alkalinity in the overtreated water will rise to 80 to 100 p.p.m. in winter, because at low temperatures not all the magnesium will be precipitated even with this excess treatment.

The return of sludge is the application of a principle that is taught to every beginner in analytical chemistry, but its use in water softening has followed the adoption of continuous sludge removal equipment such as the Dorr clarifier. For the best results, sludge should be taken from the second clarifier and not from the first. There are two reasons for this: first, sludge in clarifier No. 1 contains the great bulk of algae found in the raw water, and at times may be exceedingly foul and malodorous; second, sludge in clarifier No. 1 contains magnesium hydroxide freshly precipitated and readily redissolved if mixed with water from which excess lime has been removed by recarbonation. To check this point, we have actually used this sludge and found a rise of 20 p.p.m. in the hardness above the results with sludge from clarifier No. 2. Clarifier No. 2 contains practically pure calcium carbonate with few microörganisms or algae.

The lime feeding equipment consists of two Omega Machines designed for hydrated lime. In order to adapt these to use of quick lime, a slightly larger solution tank has been substituted for the one furnished, and an outboard stirring machine installed. With this equipment ground quick lime is successfully slaked and fed into the raw water. The saving due to the use of quick lime instead of hydrated amounts to about $5.00 per day.

Chloramine treatment

Two interesting features of the operation of this plant deserve mention, although they are not related to softening. One is chlor-

amine treatment. This is made necessary by the fact that the filtered water is stored at Bloomington in an open reservoir holding four days' supply. The treatment with chloramine carries a residual chlorine of 0.2 p.p.m. to the city taps without taste or odor during greater part of the year. During the summer months, however, biologic activity becomes uncontrollable by this method in the city reservoir, and it is then necessary to chlorinate the final high service pump suction. This is accomplished with an automatic Wallace & Tierman A. S. V. Machine. The point of application of the chloramine is in the first mixing tank so as to eliminate algae growths in the basins at the plant and to prolong filter runs.

Activated carbon use

The second feature is the use of activated carbon in taste removal. The carbon used is "Nuchar No. 2 Fine." It is applied in the second mixing tank. The carbon which settles in clarifier No. 2 is returned to the mixing tank repeatedly. Some, of course, goes into the filters and is lost in the waste water. The carbon undoubtedly removes some algae odors and will probably be used during periods when they are noticeable in the raw water. Whether it will remove all odors when used in this manner is doubtful. Our experience of less than a month warrants a qualified endorsement.

We have felt gratified at the performance of this plant which is due to a number of factors in addition to the features of design already mentioned. Uniformity and regularity are particularly necessary with this type of treatment, and they have been worked out to a marked degree. Some of the aids in this respect are: a recording thermometer placed on the lime slaker by which a constant application of lime is assured; a recording altitude gage on the applied water by which we have been able to eliminate fluctuation in raw water pumpage; the smooth and constant operation of carbonating equipment which is oil fired and requires a minimum of attention. Of equal importance may be mentioned the capable and conscientious effort of the plant operators under the direction of Delmar Riser. Acknowledgment should be made, also, of the guidance and support of C. C. Williams, Superintendent of Bloomington Water Company, and of the Bloomington Water and Light Department.

AN ARTIFICIAL SUBTERRANEAN WATER SUPPLY SYSTEM

By Leland W. Parr[1]

A clean and adequate water supply has been one of the desiderata various civilizations have striven to obtain. This was true long before the real hygienic significance of good water was known. Roman culture has contributed much to public health. It is usually credited with street sewers or cloacae, public baths, paved streets, some idea of garbage collection and disposal, and with extensive and expensive public works designed to afford a plentiful, sparkling water supply. The Romans often brought water long distances by means of aqueducts, remains of which are still to be seen stretching across the Roman Campagna. Some of the old aqueducts are still in use in various places almost exactly as they were centuries ago.

In the northern part of Arab Asia, however, there are remains of ancient means of obtaining water less familiar to the western engineer than the aqueducts of the Romans. Some of these systems may date back to the time of Queen Zenobia's great kingdom at Palmyra in the third century of this era. Some of them are still usable, although Palmyra itself is today only a collection of Arab hovels in the midst of which are a would-be modern tourist hotel and a French military post, all in a setting of acres and acres of columns, capitals and other relics of past glory.

There are snow-capped mountains in the western part of this area from whose slopes issue numerous springs of excellent quality and sufficient in quantity for the use of good sized villages and even cities. Thus Damascus, a city with a population of a third of a million, has its drinking water piped to it from a distant spring. But as the mountains level off into the plain to the east and become the Syrian desert these springs are no longer found. To attempt to divert a small mountain stream even a few miles over the surface of this area would be futile because of rapid evaporation and a soil not adapted to ditches because of lack of clay. For example, the so-called Seven

[1]Formerly of the Department of Bacteriology and Hygiene, American University of Beirut, Beirut, Syria; Andalusia, Ala.

Rivers of Damascus, whose waters make it possible for that city to be surrounded by rich vegetable gardens and fruit orchards, and which have themselves arisen in the hills to the west of Damascus, shortly are lost in the desert a few miles to the east. Formerly iron pipes were unknown and for some reason stone aqueducts were not made use of in this part of Syria, although they have been and still are used in other parts of the country.

To meet the water needs in towns situated along the edge of the desert the Syrian engineers of the past constructed artificial subterranean streams running from the foothills considerable distances to the villages in the arid areas. (See figure 1.) These streams were built by sinking a row of wells along the route of the proposed sub-

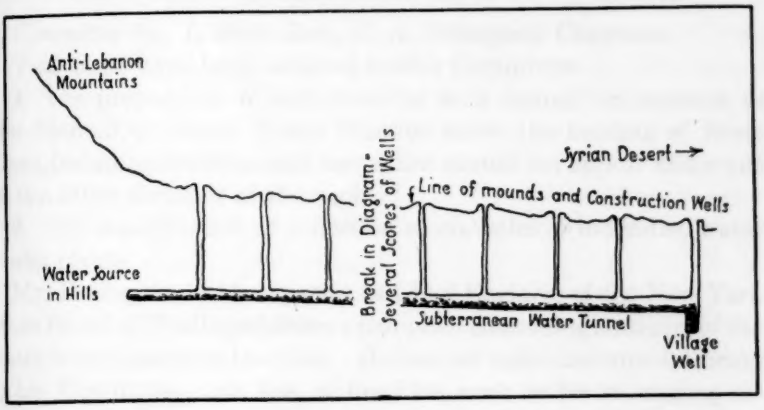
Anti-Lebanon Mountains
Syrian Desert →
Line of mounds and Construction Wells
Break in Diagram - Several Scores of Wells
Water Source in Hills
Subterranean Water Tunnel
Village Well

FIG. 1. DIAGRAM OF A SUBTERRANEAN WATER SUPPLY SYSTEM

terranean stream and removing the earth between the wells, or manholes, at a level where the flow of ground water overlay a relatively impervious stratum, thus forming a sort of water tunnel.

As one rides through such an area one sees the row of wells. They look like little mounds of earth occurring in a line about every thirty to forty yards. On closer inspection the well in the center of the mound through which the earth excavated below was removed and needed repairs made becomes visible.

Considerable skill must have been used to locate a proper source of underground water at the foothills and to preserve the correct level of tunnel excavation along a row of wells several miles long. The amount of labor involved must have been tremendous. This is all the more remarkable when one considers the lack of adequate en-

gineering apparatus and labor saving devices in use at the time the subterranean channels were constructed. Of course the daily per capita water consumption in this part of the world has always been less than we are accustomed to think of as necessary, but the amount of water used by a village of several hundred people and their animals would be none the less considerable.

ANNUAL REPORT OF COMMITTEE ON WATER WORKS PRACTICE, FOR THE YEAR 1930

Meetings held during the year

January 16, 1930, at New York.
June 2, 1930, at St. Louis.
June 4, 1930, at St. Louis.

COMMITTEES ON THE MANUAL OF WATER WORKS PRACTICE

Committee No. 1, Basic Data, C. A. Holmquist, Chairman.

Two tasks have been assigned to this Committee.

1. The preparation of such material as is desired for inclusion in the Manual of Water Works Practice under the heading of Basic Data, including statistics and text which should not appear under one of the other divisions of the book.

2. The consideration of collection of statistics of individual water works plants.

Mr. Holmquist in his capacity as Chief Engineer of the New York State Board of Health publishes a pamphlet containing statistics of the water works plants in his state. He has not made any appointments to his Committee, but has confined his work so far to making requests to the chief engineers of the state boards of health throughout the United States for copies in duplicate of such similar pamphlets as they publish. As these come in Mr. Holmquist will keep one on file at his office and send the other copy of each one to the A. W. W. A. headquarters office at New York. He has received replies from over half of the states. Much of these data changes continually. In New York he publishes his pamphlet every five years keeping the data corrected to date between times on large typewritten sheets. Some states reported to him that they have issued no such pamphlets.

Committee No. 2, Surface Water Allocation. No chairman appointed.

Committee No. 3, Surface Water Collection, W. E. Fuller, Chairman.

No other appointments made.

Committee No. 4, Ground Water Collection, D. W. Mead, Chairman.

No other appointments made.

Committee No. 5, Quality and Treatment, Paul Hansen, Chairman.

Mr. Hansen has received drafts of text for the Manual for 18 out of 29 topics from about sixty men on his committee. He will wait until he receives the other drafts before editing them, which will begin early in 1931. He will then return these drafts to the authors with instructions for revision and this material will be ready for publication in the Journal of the Association about the middle of 1931 for information and criticism of the members.

Committee No. 6, Power and Pumping, F. G. Cunningham, Chairman.

Work under way on text for the Manual.

Committee No. 7, Transmission and Distribution, G. Gale Dixon, Chairman.

This Committee is carrying on its work through twelve sub-committees of which six are organized and working.

Sub-Committee 7-A, Steel Plate Pipe for Water Supply, F. A. Barbour, Chairman.

This Sub-Committee held an organization meeting in New York in July, 1930, and adopted a program of work to include gathering history of large steel plate pipe lines used for water supply in this country, the preparation of specifications for such pipe lines as well as the writing of text for the Manual on this topic.

Sub-Committee 7-B, Reinforced Concrete Pipe.

No appointments made.

Sub-Committee 7-C, Cast Iron Pipe.

No appointments are to be made at present.

Arrangements were made this year by Mr. Dixon with A.-21, the Sectional Committee functioning under the American Standards Association on Specifications for Cast Iron Pipe and Special Castings, for the latter committee to prepare text for the Manual on this topic.

Sub-Committee 7-D, Laying Cast Iron Pipe, E. G. Bradbury, Chairman.

This Sub-Committee organized during the year and issued a program for work on its topic.

Sub-Committee 7-E, Valves, Sluice Gates and Fire Hydrants, W. R. Conard, Chairman.

This Committee organized during the year and prepared tentative specifications for valves.

Sub-Committee 7-F, Service Connections and Meters, James E. Gibson, Chairman.

This Sub-Committee organized during the year, prepared and sent

out a five page questionnaire on Service Connections and Meters to over five hundred selected water superintendents in the United States and Canada, received replies from over 50 percent of these superintendents and has prepared a summary of these replies on a set of large drawings. The Sub-Committee is now carrying on an analysis of this summary so as to get the greatest possible value from this information.

Sub-Committee 7-G, Location Records and Maintenance of Mains and Services, Albert S. Hibbs, Chairman.

No other appointments made.

Sub-Committee 7-H, Steel Standpipes and Elevated Tanks, L. R. Howson, Chairman.

This Sub-Committee organized during the year and prepared tentative specifications for Steel Standpipes and Elevated Tanks. Mr. Howson presented this specification at the meeting at Youngstown, Ohio, in September of the Central States Section, where much valuable discussion was contributed. At or soon after the end of the year this specification was in corrected form for letter ballot by the members of this Sub-Committee.

Sub-Committee 7-J, Distribution Reservoirs.

No appointments made.

Sub-Committee 7-K, Water Consumption.

No appointments made.

Sub-Committee 7-L, Fire Prevention and Protection.

No appointments made.

Sub-Committee 7-M, Hydraulics of Distribution System.

No appointments made.

Committee No. 8, Cross Connections, E. Sherman Chase, Chairman.

Committee organized and preparation of program under way.

Committee No. 9, Relations to Consumers and Public, Howard S. Morse, Chairman.

This Committee organized during the year and the Chairman has submitted to the members a tentative rating schedule for various topics of vital interest to the subject work of this Committee.

Committee No. 10, Valuation, Rate Making and Taxation, S. B. Morris, Chairman.

No other appointments made.

Committee No. 11, Office Management, Records and Accounting, D. C. Grobbel, Chairman.

No other appointments made.

Committee No. 12, Legal.

No appointments made.

STANDING TECHNICAL COMMITTEES

Standard Methods of Water Analysis, Jack J. Hinman, Jr., Chairman.

Mr. Hinman reports in a letter dated January 17, 1931, as follows:

"Following the meeting in St. Louis and in accordance with the instructions developed there, an effort was made to secure definite preparation of the material to be submitted for consideration at the time of the meeting of the American Public Health Association late in October at Fort Worth. Material was prepared on the subject of the determination of phenols by Mr. John R. Baylis, of Chicago, and transmitted to the American Public Health Association through Dr. John F. Norton, Secretary of the Laboratory Committee. Difficulty was found with the method on account of the fact that, where rather large quantities of phenols are present, the amount recovered during distillation is less in proportion than when smaller quantities are being worked with. This is important in connection with this method, because not all of the phenol is distilled over under optimum conditions and a factor has to be applied to the apparent result. The method is therefore being re-worked.

"Mr. R. C. Bardwell rewrote the material on the rapid examination of water for boiler purposes which appeared in the Journal of the American Water Works Association for August, 1925 (Volume 14, Pages 107–115). This material was circulated through Committee No. 1. and a number of suggestions were made. Thereupon the Chairman of the Committee rewrote the material and submitted to Mr. Bardwell for his approval which was given. The material was then submitted to Dr. Norton and received approval of the Laboratory Committee of the American Public Health Association. The material will probably appear as written in the forthcoming edition of Standard Methods of Water Analysis.

"The procedure for the use of brilliant green lactose peptone bile in the examination of organisms of the colon group was undertaken by Mr. Harry E. Jordan, who has been prominently connected with this work in the activities of Committee No. 1. This material was also circulated through Committee No. 1. and some errors were detected. These were corrected and the material was submitted to the American Public Health Organization. The material was approved except

insofar as interpretation of the results was concerned. It is held that the book, Standard Methods of Water Analysis, is not expected to go into matters relating to interpretation of results, but is limited to the presentation of approved procedures.

"The determination of hydrogen sulfide in waters and sewages is not in satisfactory condition and, on that account, investigations have been made on the use of the antimony tartrate method for the determination of hydrogen sulfide. The work has largely been conducted by Mr. Harry E. Johnson, Chemist in Charge, Sewage Works, Flint, Michigan, in cooperation with others who have assisted our Field Representative, Mr. Linn H. Enslow, in this type of work. Difficulty was experienced from the flocculation of the antimony sulfide. Consequently the Chairman of the Committee suggested the use of gelatine as a protective colloid. This has given fairly satisfactory results. When the matter was broached at the meeting of the American Public Health Association it was felt that this should be postponed for a time on account of the fact that the Federation of Sewage Works Associations is anxious to cooperate or initiate on its own a new set of procedures for use in the examination of sewage and sewage plant effluents. It is hoped that activity on this determination will go forward promptly. Mr. Enslow has recently prepared mimeographed directions for the use of the method as it now stands and these are being circulated through a group of sewage works analysts who may be expected to give the method proper study.

"Some time ago Dr. A. M. Buswell called the Chairman's attention to the fact that the colorimetric standards for the determination of iron were not satisfactorily worked out at the present time. Procedures originating with Dr. D. D. Jackson, of Columbia University, have apparently been altered in the course of succeeding editions of the Standard Methods. Accordingly, with the assistance of one of his students, the Chairman has undertaken to review and rework the determination of iron. A paper on this subject detailing a part of the investigation was read at the meeting of the Missouri Valley Section of the American Water Works Association in November. Only abstracts of the paper could be read, however. Subsequent work on interfering irons has been conducted and will be reported in due time.

"It has been found that adjusting the hydrogen ion concentration of the liquid in which the color of ferric thiocyanate is developed allows a much darker color to be produced with the same quantity of iron

when compared with the standards prepared by the original procedure. This seems desirable, as it should make it easier to detect minute quantities of the metal. Trials outside of the laboratory at Iowa City have indicated that satisfactory results are to be obtained by others with this method. Further trial, however, is necessary before definitely recommending the procedure for inclusion in Standard Methods.

"Mr. Lewis I. Birdsall, a Field Representative of our Committee, has also been collecting procedures for the examination of aluminum sulfate and other chemicals which have been used in the treatment of water. Just what use will be made of this material in connection with the forthcoming edition of Standard Methods remains to be developed.

"Committee No. 1. has been much interested in the Noble cyanide citrate agar which was developed at the City Board of Health of Chicago. Investigations of the material are under way at a number of laboratories, including that under the supervision of the Chairman of Committee No. 1. It is hoped that the medium may be made more suitable to less completely equipped laboratories in the future. Unfortunately, developments in connection with the City Board of Health in Chicago have caused the discontinuance of research work on this important medium. It is hoped, however, that the work may be continued again before long.

"The report of Committee No. 1 as prepared and read at the meeting in St. Louis needed extensive revision. Complete agreement as to this matter has not been reached by Committee No. 1 as a whole. Consequently the matter has been delayed somewhat. Much of the delay has been due to the fact that the Chairman of the Committee has been working under considerable disadvantage during the summer, fall, and early winter. It is hoped that considerably more progress may be made within the next month or two.

"Water laboratories in general, and especially throughout the middle west, have been very much over-burdened during the summer and fall of 1930. This has been due to the hot, dry summer, the low precipitation, and the consequent fear of many individuals and water departments about their water supplies. This fear, which was fully justified in many cases, has resulted in large numbers of examinations being demanded. The low water in many streams has afforded an exceptionally favorable condition for studies on streams undergoing minimum flows and this, too, has resulted in large amounts of labora-

tory work. The load on the laboratory under the supervision of the Chairman has been practically three-fold that normally met at the corresponding time of year.

"The Chairman of Committee No. 1. has no suggestions to make at this time concerning alterations of personnel. Perhaps such alterations of personnel may be desirable before the meeting in Pittsburgh in May."

REPRESENTATIVES ON AMERICAN COMMITTEE ON ELECTROLYSIS

Mr. Charles F. Meyerherm reports in a letter dated January 2, 1931 as follows:

"Replying to your inquiry of December 30, I would say that during the past year, the American Committee on Electrolysis has been inactive. It has held no meetings and undertaken no work.

"Regarding the activities of the Electrical Committee of the National Fire Protection Committee, I am not in position to report, because the Water Works Association has not been appointed to this committee. Representation on the Grounding Sub-Committee of the Electrical Committee has been offered to the Water Works Association, but this offer has not been accepted, pending the outcome of negotiations concerning active coöperation between the National Electric Light Association and the American Water Works Association on Grounding and other related matters."

BOILER FEEDWATER STUDIES COMMITTEE, SHEPPARD T. POWELL, CHAIRMAN

Mr. Powell reports in a statement dated October 20, 1930, as follows:

The work of the Committee during the past year has not been active owing to the illness of the Chairman extending over a period of several months.

Considerable data have been accumulated on the question of embrittlement, but up to the present time this has not been assembled. A number of failures have occurred in boilers operating with unbalanced waters, but no failures have come to the attention of the Committee where the ratios recommended by the Boiler Code Committee have been maintained.

One of the most serious difficulties that has come to the Committee's attention is that developing at some stations as a result of carry-over of silica from the boilers and depositing on the turbine blading. In

two plants this carry-over has been selective and the deposits consisted very largely of silicon oxide with only a small contamination of other salts.

An effort is being made to have the U. S. Geological Survey study the condition under which various forms of silica occur in waters, since there is a dearth of information on this subject. This is a fruitful field of research and much can be done by the individual companies in studying the form in which silica is present in concentrated boiler salines and in concentrated water in evaporators.

Information collected this year clearly indicates the necessity for better control of evaporators and improvement in design to prevent contamination of the evaporator condensate. Attention is directed to the necessity of pre-treatment of evaporator feedwater and the value of continuous blowing down. The data assembled indicate that pre-treatment and continuous blowing down of the evaporators warrants such treatment to improve the quality of the evaporator water.

The Committee is at the present time assembling data on the relative value and efficiency of the chlorination of condenser cooling water. This investigation includes the following:

1. Relative value of direct and solution feed of chlorine.
2. The influence of chlorine in accelerating corrosion.
3. The effect of chlorine in graphitizing cast iron.
4. The formation of copper chloride by chlorine treatment and a possible influence of this compound as a corrosion accelerating agent.

Consideration is also being given to the electrolytic process for the prevention of slime on condenser tubes.

The Committee finds that there is a general trend towards the use of combination treatments such as partial chemical softening followed by zeolite treatment and pre-treatment of evaporator feeds. Both of these processes indicate that under special conditions the double treatment is warranted.

The contacts made last year with foreign associations and engineers interested in boiler feed water treatment has resulted in securing a series of papers presented during the past year before Associations in America affiliated in the boiler feedwater studies.

It is the desire of the Committee to submit during the coming year reports on the data assembled concerning evaporator operation, combination softening system, corrosion studies and recent developments in prevention of embrittlement.

A further statement from Mr. Powell dated January 9, 1931 is as follows:

"My attention has been drawn to the statement in the Progress Report of the Boiler Feed Water Studies Committee, 1929–1930 which appeared in the December 6th issue of the A.S.M.E. News Bulletin. The statement referred to is as follows:

> Data collected by the Committee this year indicate that there is a general trend towards the use of combination treatments such as partial chemical softening followed by zeolite treatment and pre-treatment of evaporator feeds. Both of these processes indicate that under special conditions the double treatment is warranted!

"The statement in question was inserted in the report to indicate that consideration was being given, in the design of boiler feed water systems in specific instances, to the value of combination treatment generally and was not intended to convey the idea that numbers of such systems had been designed and constructed in this country. In fairness to manufacturers of various types of systems, there should be included in this list, combination systems consisting of chemical softening by either hot or cold lime soda processes in conjunction with phosphates, sodium aluminate and other chemicals which have proven advantageous in specific instances. These latter processes have been so widely used that it did not appear to the writer to be necessary to mention them specifically."

REPRESENTATIVES ON COMMITTEES OF THE AMERICAN STANDARDS ASSOCIATION

A-21, Cast Iron Pipe and Special Castings.

This Sectional Committee has 37 members including 5 representatives from the American Water Works Association who are T. H. Wiggin, General Chairman of the Committee, F. A. Barbour, W. W. Brush, W. C. Hawley, and E. E. Wall. A. V. Ruggles acts as Executive Assistant to the Chairman.

A previous report of the work of this Sectional Committee is given on page 649 of the May, 1930 Journal of the A. W. W. A. During 1930 work was completed on programs so far adopted for tests on cast iron pipe and fittings at three universities as follows.

> At the University of Illinois under the direction of Professor M. L. Enger strength tests were made on 6 and 20 inch pit cast pipe and on 6 and 12 inch tees, crosses, quarter bends

and wyes. These tests have added valuable and needed knowledge on the strength properties of pipe and fittings and in the case of the latter are showing how slight changes in the dimensions at important points will contribute a material advantage in strength.

At Iowa State College under the direction of Mr. W. J. Schlick, Drainage Engineer, trench load tests were completed on 20 inch pit cast pipe. Some of these tests are made with no internal water pressure and others with internal water pressure running up to about 1400 pounds per square inch. The reports on these tests offer a method of determining the allowable supporting strength of cast iron pipe subjected to internal water pressure and present average strength ratios for determining the supporting strengths with different pipe-laying conditions.

At Cornell University, under the direction of Professor E. W. Schoder, tests were completed on friction loss through 6 and 12 inch eighth bends, quarterbends, tees and crosses. These tests have been made with entry velocities varying from 3 to 10 feet per second and the fittings tested have been of two kinds, American Water Works Association standard fittings and others of shorter dimensions which the committee has under consideration. These tests are bringing out the effects on friction loss of the radius of curvature of the inside corners of tees and crosses as this inside radius is six inches in the case of American Water Works Association fittings and two and a half inches in the short fittings. Some other fittings have also been tested with one inch radius at these inside corners and still others with sharp intersections such as are found in fittings used for steam.

Tests on organic coatings and linings were made at the shops of the American Cast Iron Pipe Company at Birmingham, Ala., under the direction of Mr. S. R. Church, Chairman of Sub-Committee 3-B on Organic Coatings.

Tests in service of cement lined pipes were inaugurated at the works of the Birmingham Water Company, Birmingham, Ala., by Mr. E. O. Sweet, Superintendent of Water of that Company and Chairman of Sub-Committee 3-C on Inorganic Coatings and Linings.

Sub-Committee 3-C, E. O. Sweet, Chairman, also adopted tenta-

tive specifications for Cement Linings for Cast Iron Pipe and forwarded them to Technical Committee 3 on Corrosion and Coatings for its consideration.

The comprehensive program of tests of various kinds being carried through by this Sectional Committee results in a great deal of new information on the properties of pipe and fittings, upon which revisions of existing specifications will be based, and all of which is now being carefully studied and analyzed by the three main Technical Committees and the fourteen sub-committees handling the various topics, comprising a total membership of one hundred men actively engaged on this work.

A-35, Manhole Frames and Covers, F. A. Marston, representative of the A. W. W. A. reports as follows in a letter dated January 6, 1931:

"During the year 1930 the Sectional Committee on Manhole Frames and Covers, A-35, has had in preparation specifications for materials to be used in the fabrication of manhole frames and covers, and has also been preparing a complete set of plans showing the proposed standard designs for such castings.

"I understand that this material will be completed in preliminary form within a few weeks and will then be sent out to the members of the committee, to manufacturers and to users, to get suggestions and comments. Real progress is being made with the probability that the proposed designs may be prepared as standards for adoption in the near future."

**A-40, Plumbing Equipment,* W. C. Groeniger, Chairman. W. S. L. Cleverdon, Representative of A. W. W. A.

At the time of the organization of the Sectional Committee on Standardization of Plumbing Equipment in August, 1928, six Sub-Committees were appointed. This spring it was found desirable to appoint two more Sub-Committees to develop the sections on Copper Water Tube Fittings (No. 7) and Cast Iron Soil Pipe and fittings (No. 8). The work of organizing these two Committees and the Subgroup on Cast Iron Traps has been carried on during the spring and summer with the result that meetings of all three will be held very shortly.

Sub-Committees: No. 1 on Plumbing Code, No. 2 on Staple Vitreous Chinal Plumbing Fixtures, No. 3 on Staple Porcelain (All

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Clay) Plumbing Fixtures, and No. 4 on Enameled Sanitary Ware have the same personnel as the Standing Committees on the same subjects organized by the U. S. Department of Commerce.

Sub-Committee No. 5 on Traps. A. R. McGonegal, Chairman. The organization meeting of the Sub-Committee on Traps was held in January, 1930. At that meeting the necessity for the appointment of Subgroups to develop the proposals for traps in the following seven classes was made evident: Brass lavatories and sinks; brass; cast iron; lead; vitreous china; miscellaneous traps; interceptors and catch basins. The subgroup on Brass Lavatories and Sink Traps met in New York on March 18. The other Subgroups, while not completing their proposals, have reported progress.

Sub-Committee No. 6 on Brass Plumbing Products, J. L. Murphy, Chairman.

The Subgroup on Compression Cocks, Washer Screws, Faucets, and Tail Piece parts has sent copies of its recommendations, based on the replies to its questionnaire to manufacturers, to the members of the Sub-Committee for comment.

Sub-Committee No. 7 on Copper Water Tube Fittings. The proposal on Brass Fittings for Flared Copper Tubes is the first of the Sectional Committee's proposals to be set in type. Copies will shortly be distributed for general criticism and comment.

**B-2, Pipe Threads,* E. M. Herr, Chairman. R. D. Hall and W. W. Brush, Representatives of A. W. W. A.

Within the twelve months of the past year two of the Sub-Committees of the Sectional Committee on Pipe Threads held meetings. The progress made at these meetings is recorded in the following paragraphs.

Sub-Committee on Taper Pipe Threads. S. B. Terry, Chairman. In April, 1930, this Sub-Committee held its first meeting. The relation between the A. P. I. Standard Line Pipe Thread and the American Standard was thoroughly discussed and a Subgroup was appointed to confer with the A. P. I. Committee in Chicago next November. Consideration was then given to the content and form of the American Standard pamphlet (B2-1919) and it was decided to make no change in the form of thread, taper, pitch, and terminology.

Sub-Committee on Plumbers' Threads. L. A. Cornelius, Chairman. At its organization meeting in October, 1929, the Committee

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discussed at length the specifications for (1) Plumbers Fine Threads and (2) Corporation and Cutoff Cocks. Subgroups were then appointed to develop the details of proposed standards for these two applications.

The Sub-Committees on Straight Pipe Threads, Screw Threads for Rigid Conduit, and Threads for Thin Tubes have held meetings previously, and are awaiting the recommendations of the Sub-Committee on Taper Pipe Threads before completing their proposals.

**B-16 Pipe Flanges and Fittings*, C. P. Bliss, Chairman. F. A. Barbour, Representative of A. W. W. A. This year the Sectional Committee completed and approved two proposed American Standards, namely, those for the 800 Lb. Hydraulic Cast Iron Pipe Flanges and Flanged Fittings and the 25 Lb. Cast Iron Pipe Flanges and Flanged Fittings. The Chairman reports satisfactory progress by the several sub-committees.

Sub-committee No. 1 on Cast Iron Flanges and Flanged Fittings. A. M. Houser, Chairman. This Committee completed during the year the proposed 800 Lb. Hydraulic Standard which in November, 1929, was approved by the Sectional Committee and subsequently submitted to the three Sponsors. The Heating and Piping Contractors National Association and the A. S. M. E. gave their respective approvals in January. The approval of the Manufacturers Standardization Society of the Valve and Fittings Industry, however, was given conditionally pending changes in the Section pertaining to Markings and Materials.

The standard for 25 Lb. Cast Iron Pipe Flanges and Flanged Fittings as proposed by this Sub-committee was approved by the Sectional Committee and the Sponsors. In September it was submitted to the A. S. A. for approval and designation as an American Standard.

The proposed standard for Ammonia Fittings was completed by the manufacturers' subgroup of this Sub-committee in May. It will be discussed at a meeting of the Refrigerating Machinery Association to be held at St. Louis in October.

Sub-committee No. 2 on Screwed Fittings. S. G. Flagg, Jr., Chairman. This Committee has approved for general distribution to industry for review and comment a proposed standard for Pipe Plugs of Cast Iron, Malleable Iron, Cast Steel, and Forged Steel. This canvass of industry will be completed early in October.

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Sub-committee No. 3 on Steel Flanges and Flanged Fittings. C. P. Bliss, Chairman. In December, 1929, this Sub-committee sent out a questionnaire concerning the performance of companion flanges and flanged equipment made to the dimensions of the American Tentative Standard for Steel Pipe Flanges and Flanged Fittings. Based on the replies received certain revisions were made in the Committee's original proposal for companion flanges. This revised draft, now in proof form, will be discussed at the October meeting of the Sub-committee. In addition a proposed Standard for Steel Base-Fittings for 250, 400, 600, and 900 Pounds per Square Inch has been completed by the Sub-committee and is to be distributed immediately to industry for comment.

The Sub-committee's new proposals for 120, 150, and 1500 Lb. Steel Pipe Flanges and Flanged Fittings are to be discussed at the October meetings.

Sub-committee No. 5 on Face to Face Dimensions of Ferrous Flanged Valves. F. H. Morehead, Chairman. The work of this Sub-committee is progressing slowly. The M. S. S. V. F. I. recently requested that the Committee exclude from its scope for the present the standardization of face to face dimensions of valves rated at 150 pounds and 400 pounds W.S.P.

Sub-committee No. 6 on Malleable Iron or Steel Brass Seat Unions. C. P. Bliss, Temporary Chairman. The manufacturers' subgroup of this Sub-Committee has held three meetings during the past year and considerable progress has been made in collaboration with the A.R.A. Sub-committee on Locomotive Design and Construction, toward formulating a proposed American Standard for 300 Lb. Ground Joint Unions and Combination Union Fittings. This proposal will soon be released by the Subgroup, at which time it will be set in type for consideration by the Sub-committee and industry in general.

Sub-committee No. 8 on Marking of Pipe Fittings. W. S. Morrison, Chairman. This Sub-committee sent out a questionnaire in December, 1929, in order to obtain an expression from industry as to whether or not markings were necessary, could be put on the fittings, and how they should be applied. To this a very fair response was received representing a great diversity of opinion. One of the sponsors of this project, the M. S. S. V. F. I., is making a similar study.

The following two Sub-committees did not meet during the year: Sub-committee No. 4 on Materials and Stresses, A. M. Houser, Chair-

man, and Sub-committee No. 7 on Rating of Pipe Fittings, H. C. Heaton, Chairman. No specific progress has been reported.

**B-26, Screw Threads for Hose Couplings*, H. W. Bearce, Chairman. D. F. O'Brien, Representative of A. W. W. A.

The Sectional Committee met in July, 1930, to review and discuss the draft of the proposed standard for Small Hose Coupling Screw Threads for All Connections having Nominal Inside Diameters of $\frac{1}{3}$, $\frac{5}{8}$, $\frac{3}{4}$, 1, $1\frac{1}{4}$, $1\frac{1}{2}$ and 2 inches, which had been drafted by a Sub-committee. Based on the discussion at this meeting a revised draft of the proposal was prepared. This is being set in type for distribution for criticism and comment prior to further action by the Sectional Committee.

**B-31, Code for Pressure Piping*, E. B. Ricketts, Chairman. F. N. Speller, Representative of A. W. W. A.

The Sectional Committee has held two meetings during the past year, one at New York in December, 1929, the other at Detroit in June, 1930. At both of these meetings the progress reports of the several Sub-committees were submitted and discussed at length.

Sub-committee on Power Piping. J. H. Lawrence, Chairman. The Sub-committee has been actively engaged in the revision of the previous draft of the Power Piping section of the Code and the redrafted portions were discussed at two meetings, one held at New York in December, 1929, and the other at Detroit in June, 1930. While considerable progress has been made, this section is not, as yet, in final form.

Sub-committee on Gas and Air Piping. A. Iddles, Chairman. The work of this Sub-committee has progressed to a point where its section of the Code is in final form except for possible changes which may be made necessary as a result of requests from the natural-gas industry. It is expected that a final draft will be completed before the meeting of the Sectional Committee in December, 1930.

Sub-committee on Refrigerating Piping. A. H. Baer, Chairman. The Sub-committee has experienced considerable difficulty in drafting the section on refrigerating piping, but has recently reported that it expects to release its first proposal for general distribution in the near future.

Sub-committee on Oil Piping. A. D. Sanderson, Chairman. Some slight modifications have been made during the past year in this

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section of the Code and these, including a section for higher temperature and pressures and some modifications in the cast-iron-pipe requirements, were discussed at a meeting held at Detroit in June, 1930.

Sub-committee on Piping Materials, F. H. Morehead, Chairman. The first meeting of this Sub-committee since its organization was held at Detroit in June, 1930. It was decided there that the Committee should abandon the plan of abstracting specifications and should report in their entirety all specifications for materials applicable to pressure piping. Progress in gathering these data has been reported since the meeting.

Sub-Committee on Fabrication Details. C. G. Spencer, Chairman. A meeting of this Sub-committee was held in June at Detroit and the progress made by the various Subgroups was discussed, considerable time being given to welding and the qualifications of welders. In June, after the Detroit meeting, a draft proposal on Fabrication of Hangers, Supports, Anchors, Sway Bracing, and Vibration Dampeners was sent to all members of the Sub-committees and Sectional Committee for consideration and criticism. The Subgroup on Joints Other than Welded distributed a questionnaire to industry during the summer and replies will be discussed at the next meeting.

**B-36, Wrought Iron and Wrought Steel Pipe and Tubing,* H. H. Morgan, Chairman. F. N. Speller, Representative of A. W. W. A.

In June, 1930, the Sectional Committee met in Detroit to consider the formula for calculating nominal pipe-wall thickness as developed by the Subgroup on Basic Formulas and proposed jointly by Sub-committees Nos. 2 and 3. The formula, which had been evolved as a modification of the Barlow Formula and favored by the A. S. M. E. Boiler Code Committee, seemed to meet the requirements of pipe-wall thickness calculations, so was approved at this meeting subject to vote by letter ballot. It was recommended that, as the next step, tentative wall-thickness schedules based on this formula should be prepared and referred to Sub-committees Nos. 2 and 3.

Sub-committee on Plan, Scope, and Editing. H. H. Morgan, Chairman. This Sub-committee met in Detroit in March and in June, 1930, for an advance discussion of the problems before the Sectional Committee and to plan for the development of this project.

Sub-committee on Pipe and Tubing for Low Temperature Service.

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J. J. Shuman, Chairman, and Sub-committee on Pipe and Tubing for High Temperature Service, H. H. Murray, Chairman. A joint meeting of these two Sub-committees was held on June 10 prior to the Sectional Committee meeting. The two Sub-committees approved the formula for pipe-wall thickness and recommended its submission to the Sectional Committee for approval. Upon the approval of this formula by the Sectional Committee the two Sub-committees began the preparation of schedules of pipe and tubing sizes and weights covering the ranges needed by industry.

G-8 Work under way. R. C. Ewry and R. S. Dean, Representatives of the A. W. W. A.

Mechanical Standards Advisory Council—A. V. Ruggles, Representative of American Water Works Association.

A meeting was held in New York December 4, 1930 at which progress was made toward completing organization of this Council and its Executive Committee.

Committee on Water Works Practice:
FRANK A. BARBOUR,
ROLLO K. BLANCHARD,
W. S. CRAMER,
F. G. CUNNINGHAM,
W. W. DEBERARD,
ARTHUR E. GORMAN,
GEORGE W. PRACY,
A. U. SANDERSON,
ABEL WOLMAN,
MALCOLM PIRNIE, *Chairman,*
A. V. RUGGLES, *Secretary.*

BOCA RATON WATER TREATMENT PLANT[1]

By H. S. R. McCurdy[2]

The town of Boca Raton, located on the Florida east coast about 25 miles south of Palm Beach, has recently constructed a plant fulfilling the most rigid requirements as to pure, soft water. Situated in a region where the available waters are very hard, high in iron and color and offensive to the smell, the continued use of untreated water did not appeal to the progressive element in the town. Despite its agricultural promise, the fact remains that, for the present at least, a large proportion of the state of Florida is dependent for its prosperity upon winter visitors. This is particularly true at Boca Raton, with the establishment of the Boca Raton Club, conceived upon a large scale and counting in its membership only those who are satisfied with nothing but the best.

The geological formation of this portion of Florida consists principally of sand, underlaid, at various depths, with a porous coral rock. This rock is frequently interstratified with layers of sand, but once bed rock is reached, a generous supply of water can generally be depended upon.

It is from this coral rock formation that Boca Raton secures its supply. The plant is located west of the Dixie Highway, where the rock formation lies about 80 to 100 feet below the ground surface, in the form of a ridge deposited at the time when this was an ancient shore line.

The Boca Raton plant is constructed for an immediate capacity of 500,000 gallons per day, arranged with a view to increasing to 1,000,000 gallons per day by the simple expedient of adding two filter units. The mixing tank, coagulating basin and recarbonating basin are sufficiently large to accommodate this increase in capacity when and if it should be made. In common with all water supplies in this part of the country, a peak load is maintained for three months out of the year, with the plant idling along at a very much reduced rate

[1] Presented before the Florida Section meeting, April 11, 1930.

[2] Chief Engineer, Philadelphia Suburban Water Company, Bryn Mawr, Pa.

for nine months. Water is aerated, coagulated and softened, recarbonated and filtered.

Two 10-inch driven wells, about 400 feet apart, were sunk to ledge rock and the casing firmly seated in the rock to prevent the entrance of sand. The coral rock is sufficiently soft to allow this to be done by ordinary well drilling methods without injuring the casing. Water stands in the well about 11 feet below the ground surface, or 3 feet above sea level. Under a draft of 500 gallons per minute the drawdown in a 10-inch well amounts to about 7 feet. The elevation of the water in the casing permits pumping by direct suction, but in

FIG. 1. BOCA RATON WATER SOFTENING PLANT

order to keep the total suction lift below 18 feet, the pumps were placed at an elevation below the surface of the ground.

Pumping equipment consists of two low and two high lift centrifugal pumps, electrically operated, and one gas-driven centrifugal pump for emergency use. Each of the electrically driven pumps has a capacity of 350 gallons per minute and the gas-driven pump a capacity of 750 gallons per minute.

Power is from the Florida Power and Light Company transmission line in the street in front of the plant and is 3-phase, 60-cycle, stepped down at the plant to 220 volts.

The low lift pumps operate against a head of 31 feet, taking suction directly from the wells and delivering water over the aerator. These pumps are driven by 7.5 h.p. squirrel cage induction motors at 1750

r.p.m. Each pump has a capacity equal to the nominal capacity of the plant as at present constructed. They are located at each of the two wells, one of which is sufficiently close to the pumping station to allow the pump to be housed in the main building, while the other is in a small pump house at the second well, 400 feet distant.

DESCRIPTION OF PLANT

Owing to the fact that the raw water is high in iron and contains CO_2 and other gases in offensive quantities, aeration is necessary. The aerator consists of a steel angle weir, 11 feet long, over which the water cascades onto two steel plates arranged in steps to give the maximum splash effect. The head consumed in aeration is 2 feet.

The water passing the aerator falls into a concrete mixing tank, 11 feet square and 12 feet deep, allowing a thirty-minute mixing period. This time is somewhat longer than is necessary, but it should be remembered that the plant is designed for an ultimate capacity of 1,000,000 gallons per day.

Agitation in the mixing tank is caused by means of a Dorr impeller agitator, consisting of a propeller blade mounted on the bottom of a vertical shaft revolving at a rate of 26 r.p.m. within a concrete cylinder open at the top and the bottom, all within the square mixing tank. The action of the propeller causes a vertical circulation of the water within the concrete cylinder and between the cylinder and the walls of the tank. The agitator is operated by a 3 h.p. motor attached to the top of the shaft.

It is at the point where the water falling over the aerator enters the mixing tank that the chemicals are applied. Dry feed machines for lime and alum are mounted on the floor inside the building in such a position and at such an elevation that they can discharge into the mixing tank through very short discharge pipes. This feature was given emphasis in the lay-out of the plant, in order to eliminate the troublesome condition inevitably resulting from long discharge pipes carrying chemicals. The lime and alum are unloaded from trucks directly into a hand-operated elevator with a capacity of 2,500 pounds, discharging the chemicals on the upper floor in close proximity to the dry feed machines. In this way, manual labor in handling chemicals is reduced to a minimum.

The mixing tank discharges into the coagulating basin through a reinforced concrete gullet around its outer edge. The coagulating basin is constructed of reinforced concrete 45 feet square and with a

9-foot 6-inch side depth, providing about six hours' retention. Water enters the basin through horizontal slots along one entire side, distribution being further equalized by a wooden baffle in front of the slots.

The bottom of the basin is slightly sloped in the shape of an inverted cone, with a discharge pipe for sludge at the apex. The coagulating basin is equipped with a Dorrco Tractor Clarifier. This machine is essentially a steel truss revolving about a central axis and fitted with scraper blades conforming to the contour of the bottom of the basin. The machine is driven by means of a 2 h.p. electric motor operating tractor wheels on rails along the outside walls of the basin. The scraper blades passing over the bottom of the basin work the precipitated sludge gradually to the center and into the discharge pipe. The movement is naturally very slow, requiring twenty-two minutes for one revolution, a speed not sufficient to cause any turbidity through disturbance of the precipitated sludge. The gentle agitation of the water in the basin, caused by the movement of the clarifier, assists materially in the formation of a floc and in the precipitation of the sludge.

The discharge pipe from the coagulating basin passes beneath the floor to the suction of a No. 4 Dorrco Simplex Pressure pump located at ground level outside the basin. The sludge pump forces the sludge through a 4-inch C. I. pipe to a waste pile located in the lot behind the pumping station at a distance of about 200 feet, where the water drains away through the sand. The operation of sludge removal is such that the amount of water wasted in the process is insignificant.

After coagulation, with its attendant softening, the water passes over a weir extending along the opposite side of the basin from the influent slots, discharging into a concrete gullet leading into the carbonating basin.

Recarbonation

The carbonating basin is a concrete structure of sufficient size to provide one hour retention at the present capacity of the plant. On the bottom of this basin is a grillage consisting of a 2-inch inlet pipe with $\frac{3}{4}$-inch laterals on 2-foot spacing. The laterals are tapped with $\frac{1}{16}$-inch holes on 2-foot centers for discharging the CO_2 gas. The gas bubbles rise through $9\frac{1}{2}$ feet of water and effectually recarbonate the softened water.

The CO_2 gas is generated in a Flinn and Dreffein magazine type hot water heater with combination scrubber and dryer. The scrubber and dryer consists of a steel tank lined with brick containing trays of broken limestone and excelsior. The gases feed through the scrubber from the heater and are picked up by an air compressor and forced into the pipe grillage in the carbonating basin. Coke is used for fuel in the heater for generating the gas.

After recarbonation, the water passes directly into the filters. These are of the rapid sand type, each unit designed for a capacity of 270,000 gallons per day, figured at a rate of 2 gallons per square foot per minute.

Filter equipment

Filter equipment is the standard type of the Roberts Filter Manufacturing Company. The piping is designed for four filters, symmetrically arranged. At present only two of the filters are constructed, the piping being blanked off on the opposite side.

Collection of filtered water and distribution of wash water are by means of cast iron pipes upon the floor of the filter, consisting of an 8-inch manifold with 2-inch laterals on 6-inch centers. The bottoms of the lateral pipes are bored for $\frac{3}{16}$-inch brass eyelets, $4\frac{1}{2}$ inches on centers and staggered each side of the vertical axis, the angle between holes being 45°.

The filtering material consists of 18 inches of gravel and 27 inches of sand. The gravel is graded as follows:

Bottom layer—6 inches, passing a screen having a 2-inch mesh and retained on 1-inch mesh.
Second layer—5 inches, passing 1-inch mesh and retained on $\frac{1}{2}$-inch mesh.
Third layer—4 inches, passing $\frac{1}{2}$-inch mesh and retained on $\frac{1}{8}$-inch mesh.
Top layer—3 inches, passing $\frac{1}{8}$-inch mesh and retained on $\frac{1}{16}$-inch mesh.

The sand was specified to be of an effective size between 0.38 and 0.45 mm. and to have a uniformity coefficient not in excess of 1.65.

Wash troughs are of cast iron with the lip 2 feet above the sand, the freeboard over the troughs being 1 foot 9 inches. Water for washing filters is obtained directly from the pressure main between the standpipe and the street through a 6-inch cast iron pipe, being reduced to working pressure by means of a Ross regulating valve. Filter wash is carried off in a 12-inch cast iron pipe and discharged in the same waste area as the sludge from the coagulating basin. The filters

are equipped with the customary Venturi automatic type rate controllers and with indicating loss of head and rate of flow gauges. All valves are manually operated.

Directly beneath the filters, and extending below ground outside the building, is a reinforced concrete clear well receiving the filtered water. The capacity of this well is 65,000 gallons.

In the rear of the pumping station is an elevated tank 136 feet high holding in excess of 100,000 gallons.

From the clear well, water is picked up by the high lift pumps operating under a maximum head of 146 feet. These pumps are driven by 25 horse power squirrel cage induction motors at 1750 revolutions per minute and deliver directly into the main in the street

TABLE 1

Analyses of Boca Raton water

Results in p.p.m.

CHARACTERISTIC	RAW	FILTERED
Phenol alkalinity	0.0	6.0
Total alkalinity	146.0	32.0
Incrustant hardness	12.0	12.0
Total hardness	158.0	44.0
Color	12.0	7.0
pH	7.2	7.2
Chloride radicle	14.0	14.0
Iron	0.3	0.0

in front of the plant. The standpipe floats on the line and is connected to this main by means of an 8-inch cast iron pipe.

Analyses of the raw and of the treated water are shown in table 1.

Softening is effected by the use of 6.7 grains of lime and 0.6 grain of alum, per gallon. The reactions in the coagulating basin are complete to the extent that the filters are called upon to do very little work. It will be seen that the hardness is reduced to 44 p.p.m. and that the iron is entirely eliminated, with the result that a clear, sparkling, soft water, which will not stain the plumbing fixtures, is delivered to the consumers. Recarbonation is carried to the point where the chemical reactions are stabilized. Otherwise there would be danger of coating the sand grains in the filters and leaving objectionable deposits in the distribution system and in household piping.

The various substructures are of reinforced concrete. The super-

structure is constructed with reinforced concrete pilasters with vitrified tile blocks between and with an outside finish of cement stucco. The second floor and the roof are also of reinforced concrete, the whole being tied together as a precaution against damage from hurricane.

COST

The cost of the plant was $55,000.00. This does not include, however, the standpipe and one well which were already on the ground. A careful routing of the water through the various operations with a view to shortening lines of travel as much as possible resulted in a very compact plant and aided materially in keeping down the cost. One man is sufficient to operate the plant. In fact, he has time also to make repairs to the distribution system, read meters and perform other miscellaneous duties. The storage in the standpipe is sufficient to permit the supply to be operated on a one-shift basis for nine months of the year. During the winter peak, however, two shifts are necessary.

The Boca Raton plant was designed, under the supervision of the writer, by Wm. H. Crawford of Bryn Mawr, Pa. Crosby Tappan of Boca Raton, Fla., was the engineer in charge of construction and has acted in a consulting capacity on the operation of the plant.

SOUTH DAKOTA WATER SUPPLIES[1]

By W. W. Towne[2]

South Dakota has in the past derived its water supply chiefly from underground sources and only in recent years has there been a tendency toward the development of surface water supplies. This may be attributed to the fact that South Dakota is chiefly an agricultural and grazing state with very few cities and only in the last few years has there been any industrial development resulting in the growth of these cities.

Generally speaking the underground waters of South Dakota can be developed to produce a sufficient supply for small cities and as there is only one city in South Dakota having a population exceeding 30,000 there has been little necessity in the majority of instances for cities to change from underground to surface sources to obtain a sufficient quantity of water.

GEOLOGY OF WATER-BEARING FORMATIONS

South Dakota is also favorably located with respect to water-bearing formations in that the Dakota sandstone underlies the entire state and less extensive water-bearing strata are to be found in various localities. The Dakota formation outcrops in the Black Hills of South Dakota at an elevation of 3,000 to 3,500 feet above sea level as well as along the eastern slope of the Rocky Mountains farther west. From here it dips steeply to the east and outcrops again only at certain points in the extreme southeastern part of the State at an elevation of 1,200 feet above sea level. Because these outcroppings are at the higher elevations where the precipitation is relatively large and due to the fact that the mountain streams cross these outcroppings, this formation collects a large amount of water. In many instances these streams are but little more than half as large after flowing through the outcroppings. The formation varies from a few feet in thickness to several hundred feet in certain localities, being

[1] Presented before the Missouri Valley Section meeting, November 7, 1929.

[2] Director, Division of Sanitary Engineering, S. D.

quite porous near the top and more dense and massive near the bottom and throughout the eastern half of the State it lies from 300 to 1,000 feet below the surface of the ground.

The Dakota sandstone lies upon the Sioux quartzite and is overlain with an impervious shale thus making it an excellent storage reservoir. There are a few places within the State where the underlying quartzite pierces through the strata of sandstone. These places only occur, however, in a very few localities and the areas not underlain with Dakota sandstone are very small. Due to the fact that the water enters the strata at high elevations and the formation is surrounded by an impervious material, it does not lose much of the water that is absorbed at the outcroppings. When this source was first tapped at lower elevations, the water would rise above the surface of the ground. Consequently many cities and towns through the State tapped this supply to obtain flowing wells. This has reduced the head to such an extent that wells previously under a head of several feet now have to be pumped to produce a sufficient quantity of water.

Besides the Dakota sandstone the other less extensive formations furnish an adequate supply in certain localities. Among these are the Bennet sandstone found in the north central part of the State. This formation seems to be irregular, and disappears into the shales which surround it. In this same locality flowing wells have also been obtained by tapping some local sand deposit in the Pierre shale. These supplies depend upon local absorption of the rainfall and consequently fluctuate more than the wells in the Dakota sandstone.

The chalk stone found in the Niobrara formation lying above the Dakota sandstone outcrops along the James River in the southern part of the State and this furnishes certain localities with ample water. This water does not rise to the surface of the ground, but at the outcroppings there are numerous springs.

Besides the sources that necessitate the drilling of deep wells to obtain the water there are various sources from which water is obtained by shallow dug or bored wells. This type of well may be developed in local gravel deposits along the valleys of the Sioux and James Rivers and around the lakes in the northeastern part of the State. Some developments are found along various other streams including the Missouri River, but in general this stream carries so much silt that it seals its stream banks and there is not much seepage to supply the wells.

QUALITY OF SUPPLIES

Although the above formations will supply an ample quantity of water, the quality of this water from a mineral standpoint is very unsatisfactory, especially that of the deep artesian sources. The water in general is very hard, in many instances the permanent hardness being more than double the temporary hardness.

These deep well waters contain very little magnesium carbonate, but are high in calcium carbonate and calcium and magnesium sulphate. They also are high in sodium chlorides and sulphates and it is common to have the total solids content exceed 2,000 p.p.m.

In many instances it would be nearly impossible to produce a satisfactory water by treating it with lime and soda ash. The cost of removing non-carbonate hardness is nearly five times that of removing carbonate hardness and only in very few instances would the removal of carbonate hardness alone warrant the expense and to remove both carbonate and non-carbonate hardness would be prohibitory from an economic standpoint. Even if this were done the water would still be unsatisfactory for domestic use as in many instances the water already contains objectionable amounts of sodium and to increase the sodium content would render the water unsatisfactory. In a few localities there are some very soft waters, but are so high in sodium content as to be unfit for domestic use.

Another objectionable mineral found to a large extent in the ground waters is iron. This occurs in large quantities more frequently in the shallower wells, but nearly all well water analyses show the presence of iron in more or less objectionable quantities.

From a bacteriological standpoint the underground waters are remarkably free from contamination. However, the water is very hard on casings and some wells become contaminated by surface water entering through corroded casings.

The surface water sources of South Dakota are quite limited in their number and quantity. The Missouri River in crossing the State divides the State into almost equal areas east and west of this stream. It will of course furnish an abundant supply to any city located on its banks and a few cities so located are now deriving their water supply from this source. This stream is turbid at all seasons and carries a large amount of heavy silt. Consequently the life of raw water pumps is short and the raw water intakes often give difficulty.

RAINFALL

The area east of the river receives a fair amount of rainfall, the annual precipitation averaging about 20 inches. West of the river with the exception of the Black Hills the annual precipitation rarely exceeds 15 inches. As might be expected streams are few and small west of the Missouri River and are subject to a very small minimum

TABLE 1

Typical analytical results

(Results in grains per gallon)

CHARACTERISTIC	YANKTON		REDFIELD	JAMES RIVER AT HURON
	Well	Missouri River		
Sodium chloride	7.50	1.47	23.25	9.96
Sodium sulphate	3.98	8.98	98.31	23.37
Magnesium sulphate	16.91	7.58		8.92
Calcium sulphate	45.76			
Calcium carbonate	9.38	11.25	5.39	12.83
Iron and aluminum oxides	0.55	0.12	0.28	0.10
Silica	0.43	1.23	0.38	0.62
Organic and volatile matter	4.40	0.80	0.34	1.07
Total solids	88.91	31.76	137.04	60.19
Chlorine	4.55	0.89	14.11	6.05
Sulphur trioxide	40.41	10.10	55.41	19.12
Calcium oxide	24.10	6.31	3.02	7.19
Magnesium oxide	5.67	2.69	1.46	4.58
Iron and aluminum oxides	0.55	0.12	0.28	
Silica	0.43	1.23	0.38	
Total hardness calculated as $CaCO_3$:				
Calcium oxide, $CaCO_3$	43.00	11.25	5.39	12.83
Magnesium oxide, $CaCO_3$	14.07	6.69	3.63	11.36
Total hardness, $CaCO_3$	57.07	17.94	9.02	24.19

flow. The streams east of the river are also comparatively few and are not of sufficient size to furnish adequate water near their sources. The two streams of chief importance are the Sioux and James Rivers. These streams are very sluggish and their valleys are flat. To create any storage facilities would necessitate the flooding of large areas of land and then the reservoir would be so shallow that difficulties would no doubt be encountered in operating a filtration plant.

SURFACE SOURCES

In the northeastern part of the State are several lakes which will furnish ample water to cities located near these bodies of water. The streams in the Black Hills will supply ample water to all cities and towns located in that section of the State. At the present time, however, most of these cities obtain their water supply from springs near the head waters of the various streams.

With the exception of this source, all surface waters require complete treatment including coagulation, sedimentation, filtration and

TABLE 2

Seasonal variation in Vermilion River mineral content

(Results in grains per gallon)

CHARACTERISTIC	SUMMER	WINTER
Sodium chloride	1.08	3.35
Sodium sulphate	8.42	5.40
Magnesium sulphate	5.69	28.02
Calcium sulphate	0.79	41.34
Calcium carbonate	7.89	19.19
Iron and aluminum oxides	0.04	0.23
Silica	0.58	1.52
Organic and volatile matter	0.97	8.23
Total solids	25.46	107.28
Chlorine	0.66	2.03
Sulphur trioxide	9.00	45.99
Calcium oxide	4.75	27.78
Magnesium oxide	1.91	9.39
Total hardness calculated as $CaCO_3$:		
Calcium oxide, $CaCO_3$	8.48	49.56
Magnesium oxide, $CaCO_3$	4.73	23.30
Total hardness, $CaCO_3$	13.21	72.86

chlorination to make them satisfactory for domestic use. For the present supplies in the Black Hills, chlorination is all that is needed to make them safe for domestic use and due to the fact that this region is the vacation ground of thousands of tourists annually these supplies need careful attention to insure against dangerous pollution.

From a mineral standpoint the streams of South Dakota contain relatively large amounts of objectionable minerals. Probably the Black Hills streams and springs contain lesser amounts of minerals in solution than any other water supply source within the State.

The other streams resemble the ground waters in that they are also relatively high in non-carbonate hardness and also contain sodium in solution. These minerals are found in smaller quantities than in the ground waters but in the winter these streams closely resemble the shallow wells in that vicinity in mineral content. Iron is also found in objectionable quantities in several streams. This objection is easily solved where treatment works are installed at practically no added expense.

Some mineral analyses representative of the waters found in South Dakota are shown in table 1. The city water at Yankton was obtained from Dakota sandstone by a well, 632 feet deep.

Yankton, however, completed the construction of a new water treatment plant obtaining its future water supply from the Missouri River. The analysis of the Missouri River at Yankton, which is a great improvement over the well water, is also shown in table 1.

In table 2 analyses are given to show the difference in mineral content during the summer, when the Vermilion River is supplied by surface run off, and during the winter, when the river is fed by ground waters.

The author wishes to thank Mr. Guy G. Frary, M.S., State Chemist, South Dakota State Chemical Laboratory for furnishing the mineral analyses which are used in this paper.

EFFICIENT WATER WORKS OPERATION AND THE PURIFICATION LABORATORY[1]

By Martin E. Flentje[2] and P. S. Wilson[3]

Justification, as such, of the expense of maintaining a purification department with the necessary bacteriologists and chemists is not required in any water works organization. From the humanitarian standpoint alone such supervision is required to protect the water consumers' health regardless of costs. That the purification department, or chemist in charge, may be a good business proposition tying in with the general efficiency of the other water works operating departments, is, however, not so well understood; and it is with this phase of the purification department activities that this paper deals.

With this in mind it is hardly necessary to mention that, while savings and methods of increasing efficiencies are stressed in this paper, the company with which the writers are connected places ahead of any possible reduction of costs or increased efficiency, the production of pure water. Each of the companies supervised is required to produce water meeting the bacterial standards of the United States Treasury Department standards for Inter-State Carriers. To properly supervise the purification at each plant, it has increased the personnel at its central and district laboratories, has increased the number of samples examined periodically, and has in practically every case raised the standards to be met.

Next to the production of pure water, a purification department should attempt to judge its work as any technical or control department of a large chemical company would do. The production of water is a "heavy chemical" undertaking, and as such requires not only the production of a salable product, but also production at the lowest cost consistent with high quality standards. Efficient chemical

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production is always accompanied by close laboratory control, and considered in the light of a chemical industry, there is probably no other industry of like size and with like volume in which so little technical control exists as in the supply of water. This is not as true as it was, but our experience in operating a number of plants under central control, indicates there is still considerable room for improvement. Production of the sole commodity of a water company is far too often left to experienced, but not necessarily trained men, or when supervision is provided it stops far short of this "business" side of the problem, and what would ordinarily be provided in a chemical factory of like size.

COST OF PURIFICATION CONTROL

In a pamphlet published by Wallace and Tiernan Company, J. A. Tobey, makes the statement: "The review of the score or so of court decisions on the liability for typhoid is convincing proof that, entirely apart from the humanitarian aspects involved, it is poor business for a corporation of any kind to neglect such an important matter as the safeguarding of the water it dispenses." To see just how "good business" it is, we have attempted to compare the cost of purification control with similar protection against possible losses, as for instance fire insurance. The average premium fire insurance rate for 1929 was 89 cents per $100, with a burning ratio, computed on the amount of losses paid and the amount of risk, without regard to the premiums received, of 0.41. It is practically impossible to compute a similar ratio for possible losses to a water company if an epidemic resulted through negligence or lack of proper supervision. The comparison of such figures with the cost of protection against possible losses from water borne epidemics, involving as it did the lack of definite damages, incomplete health records, etc., turned out to be a much longer job than anticipated and was therefore not completed. The average water department or company can, on a comparative basis alone, afford, however, to spend considerably more than it does in improved purification control.

TECHNICAL CONTROL

The preparation of water for filtration presents an interesting field for greatly improved control. The tendency of practically every small plant operator is to rely on his eye and judgment as to what constitutes a good "floc." A good floc may, however, not represent

a good floc and a minimum alum dosage to produce it. The natural tendency of such control is to let well enough alone, and to overcome this tendency this company has found it desirable to install stirring machines in every plant laboratory where such a machine can be used. These machines with a daily jar test, requiring twenty minutes time, makes the operator, "alum minded" and insure the proper dosage with every change of water character. This does not always result in lowered costs, but it insures a properly dosed water and with much improved uniformity. The failure of "eye control" is well illustrated in a short study of the alum dosage of a Community Water Service plant, during a period of ninety-four days, where unusual

TABLE 1

Alum dosage study, Plant A, Community Water Service Company

	TURBIDITY RANGE—RAW WATER			
	0–50 p.p.m.	50–100 p.p.m.	100–150 p.p.m.	Over 150 p.p.m.
Alum dose, pounds per million gallons				
Average	100	165	218	320
Maximum	220	260	310	600
Minimum	32	65	120	160
Number of days this range turbidity present	25	37	14	18
Maximum possible saving, dollars	99.00	215.00	110.00	225.00

Note: Savings based upon actual costs and possible costs if minimum dose in any particular turbidity range had been used when such turbidity was present.

conditions were entirely absent, and the turbidity present alone practically determined the alum dosage. These data appear in table 1.

This study indicated that under entirely comparable conditions a maximum possible saving of $650.00 could have been effected in a period of ninety-four days. The maximum is, of course, not attainable, but the possibilities are indicated, even in a plant of moderate size, the capacity of this plant being from 4 to 6 m.g.d. This plant now controls its alum dosage by such a stirring machine. A recent statement from the plant operator was to the effect that his plant was now producing the best water in its history, showing that other benefits than cost reductions are a part of good control. It might be added that lower costs have also resulted.

Jar tests made periodically may point out faulty settling basins, or improper mixing, as laboratory dosage tests made under same temperature conditions should approximate plant results. In the operation of several plants, such jar tests have clearly indicated that new improvements for mixing and settling would not only pay a good rate of return on the investment required, but also help produce a more satisfactory water.

In another plant belonging to this company, a method of using what is usually an undesirable supply, has been worked out. This company has two supplies, one of good quality, but insufficient in quantity, and another of large quantity, but carrying a high content

TABLE 2

Analyses—utilization of water bearing acid mine wastes for coagulation

(Results in parts per million)

CHARACTERISTIC	BEAVER RUM WATER CONTAINING MINE WASTES	GILKERSON RESERVOIR (OF GOOD QUALITY)	MIXTURE AFTER COAGULATION AND FILTRATION
Color	5	12	
Turbidity	90	30	0
Alkalinity	−49	16	27
Total hardness	194	70	94
Free CO_2		2	
Iron (Fe)	13.0	0.74	0
Nitrite N	0.002	0.026	
Nitrate N	0.30	1.0	
pH		6.8	7.8
Iron sulphate	35.3		
Aluminium sulphate	106.6		

of acid mine water. This stream has had a content of total acidity as determined in a hot solution with phenolphthalein indicator of 150 p.p.m. (as $CaCO_3$). Alum, and sometimes, lime is used to treat the smaller (Gilkerson supply) and caustic soda, or lime and soda ash, is used to treat the supply carrying mine waste. The acidity is due to some free acid, and a large quantity of iron and aluminium sulfate. Mixing the two supplies and adding sufficient lime to bring the pH to a point insuring precipitation of the iron and aluminium sulfates has not only eliminated the addition of alum when the turbid soft water is being treated, but, more important, it will prolong the time the good supply can be used and necessarily shorten the time the ob-

jectionable mine waste bearing water must be treated. Typical partial analyses of these supplies, and a mixture are given in table 2.

In another plant, whiting has been used for a number of years as an aid to color removal. The introduction of whiting to this particular plant solved a very difficult problem that had baffled a number of prominent purification men, during the first few years of operation. In connection with an investigation of the possible uses of water softening sludge, it was found that this sludge could successfully be substituted for the whiting. Freight rates and drying costs practically eliminated the use of this material, but indicated the possible use of a by-product of the lime industry—"limestone flour." In composition, the sludge and limestone flour are very similar, and in actual

TABLE 3

Materials used as aid in color removal at Plant C, Community Water Service Company

	SLUDGE OF WATER SOFTENING PLANT AFTER DRYING	WHITING	LIMESTONE FLOUR
Moisture, per cent	1.03	0.30	
Insoluble	1.93	8.17	0.84
Oxides of Fe and Al	1.60	0.50	0.70
Calcium carbonate	81.02	90.76	85.50–91.00
Magnesium carbonate	15.41	0	10.40–13.30
Total carbonate as $CaCO_3$	99.30	90.76	
Price delivered per ton, dollars		13.90	8.90

plant operation, this product can be substituted with no apparent differences in results. Comparative analyses are shown in table 3.

With 80 tons of this material being used per year, this new material is readily accepted by the plant operator and manager.

The use of soda ash in preference to lime for the correction of low pH values in soft waters is a debatable subject upon which definite recommendations are hard to obtain. One of the main objections to lime is the additional hardness caused by it, while the main objection to soda ash is the cost. In waters having hardness from 10 to 50 p.p.m., it seems debatable that any small increase in hardness is objectionable. In one of this company's plants soda ash had been used for a considerable period of time. In order to determine the possible effect on the consumer of the slight additional hardness lime

usage would cause, it was decided to make a plant test with lime over a long period of time. This was done, and not a single complaint has been received, even though the hardness has been increased 8 to 10 p.p.m. or approximately 40 per cent. This substitution has resulted in a saving of approximately $4,000 per year, in addition to the possible benefits of lime in the pH correction of soft waters. Average results at this plant are given both before and after the substitution of the lime for the soda ash.

The possibilities in some of the other problems confronting the water works man seem very large. In the field of corrosion an enor-

TABLE 4

Comparison of soda ash and hydrated lime for pH correction at Plant D, Community Water Service Company

MONTH	RAW WATER—TOTAL HARDNESS	COAGULATED WATER		TREATED WATER			AVERAGE DOSE	
		pH	Alkalinity	pH	Alkalinity	Total hardness	Soda ash	Lime
							pounds per m.g.	*pounds per m.g.*
December, 1928	26.0	6.40	6.5	7.15	19.0	24.0	123	0
January, 1929	23.5	6.50	5.5	7.05	15.0	24.0	96	0
February, 1929	22.0	6.40	4.0	7.10	15.0	23.5	112	0
March, 1929	25.5	6.30	3.5	7.10	16.0	24.5	126	0
April, 1929	Lime and soda ash each used part of month							
May, 1929	24.5	6.15	2.0	7.90	11.5	32.0	0	52
June, 1929	23.0	6.25	2.5	7.40	10.5	31.0	0	50
July, 1929	21.5	6.20	4.0	7.15	11.0	27.5	0	48
August, 1929	20.5	6.30	6.0	7.30	13.5	29.5	0	53

mous amount of practical research remains to be done, particularly in the application of preventative measures. In our own experience this has been particularly well illustrated in a transmission main 38,000 feet in length, which requires periodical cleaning to allow its full use. This main is a 16-inch force main, one-third of it laid in 1885 and the remainder in 1925. The experiences and records in cleaning this main have been reported. For this report it is sufficient to state that, in spite of maintaining a pH value, with lime, of between 8.1 and 8.6, in a water having an alkalinity of 25 to 50 p.p.m., nodulation proceeded at such a rate as to make cleaning again necessary in 8 to 10 months. This main before cleaning had a capacity of 2.1 m.g.d.

under an increased pumping head dangerously close to the allowed limit. A loss of head of 5.38 feet per thousand feet of pipe, when the maximum capacity was transmitted, was reduced to a loss of head of 1.87 feet at the same capacity; or the capacity of the pipe was increased from 2.1 to 3.1 m.g.d., with a reduction in the initial head of 40 pounds, and an increase in residual pressure of 45 pounds by cleaning the main. When the increased costs, the possibility that other sections of the distribution system are in similar condition, and the fact that the decreased capacity of the main requires an outlay of sums up to $100 per day for purchased water, are brought out, the need for a practical method of preventing nodulation are apparent. Considerable time and money can be expended in such research.

SPECIAL USES OF LABORATORY

The laboratory analysis and test of fuels, both coal and oil, as used in steam and Diesel pump stations, may save many times the cost of the work, not only in checking the fuel as delivered to see that it conforms to specifications, but in ascertaining what specifications are the correct ones to apply under different conditions and in different parts of the country where the character of the fuel sources varies. A certain specification may secure the best fuel, but in some parts of the country the increased freight charges necessary to meet that specification may offset the advantage. Another great benefit resulting is the fact that when the analyses are made by the company's own laboratory the plants are encouraged to obtain them more frequently and more as a matter of routine. This enables them to check their plant efficiencies more accurately in other respects and leads to more careful and efficient operating all around.

The laboratory is performing valuable service in the analysis of other materials (besides chemicals) used by the various companies. Examples of this are analyses of brasses and bronzes as used in service line materials. Cast iron analyses may also be checked.

An active laboratory organization has been found to be of great value in investigating the methods of treating boiler feed water. All waters are different and no one compound or method of treatment is best for all. Analyses of the water and of the scales formed, carried on during various seasons, and under different operating conditions, may point the way to real savings in the form of operating economies and reduced maintenance. Along the same line is the opportunity to handle the treatment of scale-forming waters used for the cooling

of Diesel engines. This latter is at least as serious a problem as the treatment of boiler waters for expensive maintenance and shut downs may result from its neglect.

The broadest field by far for the activity of the chemical engineers has been found to be in the solution of corrosion problems. Every source of water supply constitutes a complete subject for study. The subject may be roughly divided into two parts: the elimination of corrosion as it affects the character of the water, for instance causing "red water" troubles, and the prevention of the harmful effects of corrosion on the plant itself, destructive corrosion of pipe materials both interior and exterior and the reduction of pipe capacities by tuberculation. The question of the exterior corrosion of pipes brings in the study of soils which in itself is an almost unlimited field.

It cannot be claimed that even a preliminary conclusion has been reached in the work on corrosion. Much very useful work has been accomplished, however. The quality of service rendered has been greatly improved in some cases and the methods of treatment formerly in use replaced by ones less costly and just as effective. Definite information is at hand as to the characters of the various waters in use and measures preventive of corrosion and tuberculation are being applied where needed. Much further work is yet to be done on this subject.

Thanks are due the various companies and their managers for permission to present these data, as well as to A. T. Clark, Chief Engineer, Community Water Service Company, for the loan of tabulations.

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DISCUSSION

A. T. CLARK:[4] The paper by Mr. Flentje and Mr. Wilson should be of exceptional interest not only to those engineers whose duty is

[4] Chief Engineer, Community Water Service Company, New York, N. Y.

mainly purification, but also to those operating engineers whose operating ratios may be bettered by a closer coöperation and contact with the laboratory.

A comparison of the chemical analyses of water before and after treatment, its analysis after period of retention in a transmission or distribution system and the analysis of the nodules or tubercles formed in the transmission or distribution pipes, opens up a field for considerable further activity by chemical engineers.

The solution for the prevention of corrosion or nodulation has only been touched upon and to bring out the saving which might be made by the prevention of corrosion, I should like to present the following figures, as furnished by one of our companies, taken before and after the cleaning of a long transmission main.

The prevention of corrosion or the cleaning of a water main or the removal of the nodulations and sediment which under ordinary circumstances are more than likely to develop, results primarily in an increased capacity of the main and in increased pressures. In the case of a transmission main carrying water from a pump station, economical advantages would also be obtained and the results of the prevention of corrosion or the cleaning, in dollars and cents, may be accurately determined where sufficient records have been kept.

In the particular instance referred to, it was found that the pumping cost prior to cleaning, was $25.50 per million gallons, as station send out. This amounted to 7.68 cents, per foot head, per million gallons. After the cleaning of this transmission main, the cost per million gallons send out, was reduced to $14.05 per million gallons or 4.25 cents per foot head, per million gallons.

In other words, aside from increasing the capacity of this pipe line from 2,100,000, under a maximum limiting pumping head, to upwards of 3,100,000 gallons per day, a saving of $10.15 per million gallons in operating cost was also effected.

Assuming that the effects of the cleaning lasted for a period of only one year, in which time the capacity was again reduced to its previous figure, which greatly underestimates the benefits derived, and in the particular case of the plant referred to, that the annual pumpage amounted to 950 million gallons, the indicated savings due to cleaning, from an operating standpoint only, would be roundly $4,800.00.

If corrosion could be prevented by laboratory control, twice the above saving would have been indicated.

Even this amazing figure does not take into consideration the advantage of being able to deliver approximately 50 percent more water, without increasing the pumping head or the greater advantage of deferring otherwise necessary capital expenditures.

L. T. REINICKER:[5] The many uses and the justification of the purification department with its trained personnel, and the placing of such a department on a business basis have been clearly demonstrated in the paper just presented. Too little thought has been given to the business side of all operations of the water plant, as we have been more or less content to struggle along and furnish water to those that desire it, and have failed to follow the policy of the other utilities in making the sale of our commodity a real business undertaking.

However, the most important function of the purification department and one which, in the writer's opinion, should be stressed, is the education, experience and training which such a department can pass on to all the employees in charge in the operation of one or more water plants.

It is conservative to state that the small plants supplying water to communities of less than 25,000 population, are in the great majority. In such plants the superintendent or the man in charge of operations, theoretically, must be a specialist in all phases of water works operations. Not only must he be entirely familiar with the problems in furnishing an adequate quantity of water, but he must also recognize and understand the important features connected with the treatment of water.

However, efficient operations can only be realized through a thorough understanding of each individual problem and how often do we find the manager of a small plant lacking in the fundamental knowledge of how to proceed with this treatment of water.

Small town operators are as a rule capable and earnest public servants, but are only prevented from emulating their big city brothers in the work they do by their own limitations. Out of contact with other men trained in their profession, it is often necessary that such men, although realizing the importance of the purification of water, are unable through lack of knowledge to apply the practice of water treatment and protect the lives and health of their consumers. In our operation of small plants we have discovered that a number of employees in responsible positions do not even appreciate the importance

[5] Operating Manager, National Water Works Corporation, New York, N. Y.

of water purification. So long as none of their consumers become ill they feel under no obligation to worry about the quality of the water they are supplying.

Such a condition is indeed unfortunate and is due entirely to the inability of such men to obtain the information they desire in such form as to be intelligible to them. In water plants of large towns there is sufficient work to justify the creation of departments, especially the purification department, composed of men thoroughly aware of the importance and science of treating water. Here salaries can be paid to attract the employment of men outstanding in this field and money can be expended in research work and study in the increasing and interesting problems in water purification. From such men in their research work contributions are made to this department of water works operation which are of extreme value and properly understood and appreciated by other such trained men. However, such information is beyond the conception of the operator of a small water plant, with the result that, as he is not trained even in the fundamentals, he is unable to derive any benefit from the experiences of the outstanding men in this field.

Through the creation of a purification department, such as that maintained by the Community Water Service Company, the personnel of the various plants controlled by the above company may be instructed and trained in the treatment of water. The benefits from this work alone are sufficient to justify the maintenance of a purification department, even if it were not also possible to develop the business side of the problem.

With the importance of purification stressed now above all other phases of water works operations and with the increasing trend toward the treatment of all public water supplies, the field is fertile for the publication of the theory and the practice of water treatment in clear, concise, practical language, which can readily be understood by the average small town operator. This, accompanied by contact with other men through short term schools conducted in various convenient locations, should materially increase the efficiency of all water works operation and will make for greater safety in the quality of water supplied.

In the small plants operated by a parent company, the operators can be trained by the central purification department, but the great majority of small plants, private and municipally owned, which prob-

ably never will be operated by a parent company, are still in a position of not having the necessary information available to understand the importance of producing pure water. This water purification division can be of wonderful assistance, therefore, to such small town operators in showing more interest in their problems and in trying to appreciate and understand their lack of knowledge of the subject.

TYPHOID FEVER IN THE LARGE CITIES OF THE UNITED STATES IN 1930[1]

This report concerns ninety-three cities of more than 100,000 population. This is an increase of twenty-five cities over the 1920 enumeration and a population increase of more than 8,500,000. Whereas in 1920 there were in the sixty-eight cities of more than 100,000 a total population of 27,735,000 persons, comprising 26 per cent of the population of the continental United States, in 1930 in the ninety-three cities of this size there were 36,326,000 persons, about 30 per cent of the total population of the country. Fifteen million of these 36 million live in the five largest cities (New York, Chicago, Philadelphia, Detroit and Los Angeles).

Twelve cities appear this year in these summaries for the first time: Chattanooga, Tenn.; Evansville, Ind.; Fort Wayne, Ind.; Knoxville, Tenn.; Long Beach, Calif.; Miami, Fla.; Peoria, Ill.; Somerville, Mass.; South Bend, Ind.; Tampa, Fla.; Waterbury, Conn., and Wichita, Kan. They are distributed among six of the eight geographic divisions, the Middle Atlantic and West South Central groups being the only ones without additions. Data for past years for all the new cities except South Bend have been supplied by the local health departments.

Divergences between previously estimated population figures for the years 1926–1929 and the populations calculated on the basis of the 1930 federal census appear in most instances to have affected but little the typhoid death rates given in previous reports. In twelve cities, where the 1926–1930 averages calculated on the basis of the rates in previous reports departed from the averages of the rates calculated by the 1930 census by 0.3 or more, the rates have been

[1] Reprinted from the Journal of the American Medical Association, 96: 19, May 9, 1931, p. 1576. Preceding articles were published in the same Journal, May 31, 1913, p. 1702; May 9, 1914, p. 1473; April 17, 1915, p. 1322; April 22, 1916, p. 1305; March 17, 1917, p. 845; March 16, 1918, p. 777; April 5, 1919, p. 997; March 6, 1920, p. 672; March 26, 1921, p. 860; March 25, 1922, p. 890; March 10, 1923, p. 691; Feb. 2, 1924, p. 389; March 14, 1925, p. 813; March 27, 1926, p. 948; April 9, 1927, p. 1148; May 19, 1928, p. 1624; May 18, 1929, p. 1674; May 17, 1930, p. 1574.

recalculated on the basis of new population estimates based on the 1930 census; the twelve cities affected are: Cambridge, Mass.; Camden, N. J.; El Paso, Texas; Jacksonville, Fla.; Los Angeles; Louisville, Ky.; Memphis, Tenn.; Norfolk, Va.; Portland, Ore.; Tulso, Okla.; Washington; and Wilmington, Del.

The number of typhoid deaths during 1930 in each city has been sent by the respective health department. The rates are calculated on the basis of the 1930 census, according to the revised figures given out by the Bureau of the Census, Jan. 17, 1931. It should be noted that the figures for typhoid deaths include all that have occurred within the city limits, nonresidents as well as residents, since statisticians are agreed that "the attempt to eliminate the deaths of nonresidents would often result in an understatement of the true mortality."[2] It is evident, however, that nonresident deaths offer an important problem in a number of localities. In four cities where only single deaths from typhoid are reported, the death in each case is that of a nonresident: Springfield, Mass.; Syracuse, N. Y.; Flint, Mich., and Grand Rapids, Mich. In 27 others, one-third or more of the typhoid deaths reported are stated to have occurred among nonresidents. In Providence, R. I., 4 out of 5; in Atlanta, Ga., 21 out of 28; in Birmingham, Ala., 8 out of 12; in Dallas, Texas, 11 out of 17; in Oakland, Calif., 2 out of 3, and in Los Angeles, 8 out of 18 are so classified. Some other cities are not far behind in having a high proportion of nonresident typhoid deaths. In view of the greatly diminished typhoid rates in most localities, this factor is bound to receive greater attention in future statistical inquiry. Typhoid rates in some parts of the United States evidently connote a regional rather than a municipal problem.

Three of the fourteen New England cities (Bridgeport, New Haven and Worcester) had no deaths from typhoid during 1930 (table 1). New Haven, indeed, has had but two deaths from typhoid in four years, a remarkable record; this city ranks third best in New England for the 1926–1930 period after being one of the "tail-enders" for nearly twenty years. The highest honor for 1926–1930 in this group belongs to Springfield, with an average typhoid rate of only 0.4 per hundred thousand (three deaths in five years). Boston is well in the upper half of the list with an average for 1926–1930 of only 1.2 and a still better showing for 1930 (0.8). The two newcomers among New England cities, Waterbury and Somerville, stand midway among

[2] Bureau of the Census, Mortality Statistics, 1912, p. 13.

their fellows; both cities seem to have had about the average amount of typhoid in the past and to have participated similarly in the general decline. Lowell brings up the bottom of the list in 1930, as it did in 1929, and even shows a slight typhoid increase. Why does the Lowell typhoid death rate continue to be nearly four times the average for New England cities (table 12)?

The typhoid rate for the New England group as a whole (1.23) is distinctly lower than that for 1929 (1.35) and with one exception (1928) the lowest ever recorded (table 12). All New England cities

TABLE 1

Death rates of cities in New England states from typhoid per hundred thousand population

	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910	1930	1929	1928	1927	1926
Springfield	0.4	2.0	4.4	17.6	19.9	0.7	0.7	0.0	0.0	0.7
Bridgeport	0.5	2.2	4.8	5.0	10.3	0.0	0.6	0.6	0.6	0.6
New Haven	0.6	4.4	6.8	18.2	30.8	0.0	1.0	0.0	0.0	2.2
Worcester	1.0	2.3	3.5	5.0	11.8	0.0	1.0	0.5	3.1	0.5
Boston	1.2	2.2	2.5	9.0	16.0	0.8	1.6	0.6	1.1	1.8
Waterbury	1.2	1.0	8.0	18.8		1.0	1.0	2.0	0.0	2.0
Hartford	1.3	2.5	6.0	15.0	19.0	2.4	0.0	1.2	1.8	1.2
Providence	1.3	1.8	3.8	8.7	21.5	2.0	1.7	1.7	0.4	0.7
Somerville	1.3	1.6	2.8	7.9	12.1	1.9	0.0	1.9	1.9	1.0
Lynn	1.5	1.6	3.9	7.2	14.1	1.0	1.9	0.0	0.9	3.8
New Bedford	1.5	1.7	6.0	15.0	16.1	2.7	0.0	0.8	3.3	0.7
Cambridge	2.1	4.3	2.5	4.0	9.8	0.9	1.8*	1.8*	0.9*	5.3*
Fall River	2.2	2.3	8.5	13.4	13.5	2.6	0.7	4.5	2.3	0.8
Lowell	2.6	2.4	5.2	10.2	13.9	5.0	4.4	0.0	2.6	0.9

* Previous figure revised on basis of 1930 census population.

save two (Waterbury, Lowell) had a lower five year average for 1926–1930 than for 1921–1925. For the second year in succession, however, the New England typhoid rate has been higher than that for the Middle Atlantic states (table 12).

The cities of the Middle Atlantic states make the best showing of any geographic group in 1930 (table 12). Philadelphia ties with New York for 1930 and actually has a lower five year average. In view of the past typhoid history of Philadelphia (for instance in 1906–1910) the improvement is amazing. Both New York and Philadelphia reduced by half their typhoid rates in 1926–1930 as

compared with 1921–1925. Perhaps the most remarkable fact about the Middle Atlantic group is that every one of the eighteen cities made a better record in 1926–1930 than in 1921–1925. Two cities, Paterson and Utica (table 2), have a perfectly clear record with no typhoid deaths, both for the second time, Paterson having a similar record in 1927, Utica in 1926. Elizabeth, after an unusually

TABLE 2

Death rates of cities in Middle Atlantic states from typhoid per hundred thousand population

	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910	1930	1929	1928	1927	1926
Yonkers	0.5	1.7	4.8	5.0	10.3	0.7	1.6	0.0	0.0	0.0
Syracuse	0.8	2.3	7.7	12.3	15.6	0.5	0.5	0.5	1.5	1.1
Erie	0.9	2.3	6.9	49.0	46.6	0.9	0.7	0.8	1.5	0.8
Jersey City	0.9	2.7	4.5	7.2	12.6	0.3	0.3	0.6	1.2	1.9
Newark	0.9	2.3	3.3	6.8	14.6	0.2	0.6	1.0	1.3	1.5
Paterson	1.0	3.3	4.1	9.1	19.3	0.0	2.0	1.4	0.0	1.4
Philadelphia	1.1	2.2	4.9	11.2	41.7	0.9	0.7	0.8	1.4	1.9
Utica	1.1	3.9*				0.0	0.9	1.9	2.9	0.0
New York	1.3	2.6	3.2	8.0	13.5	0.9	1.2	1.4	1.3	1.9
Elizabeth	1.6	2.4	3.3	8.0	16.6	4.4	1.7	0.0	0.9	1.0
Reading	1.6	6.0	10.0	31.9	42.0	0.9	0.0	2.6	1.7	2.6
Rochester	1.7	2.1	2.9	9.6	12.8	1.8	0.6	0.9	1.2	4.0
Albany	1.8	5.6	8.0	18.6	17.4	0.8	3.3	0.8	3.3	0.0
Scranton	1.8	2.4	3.8	9.3	31.5	1.4†	1.4	1.4	0.7	4.2
Trenton	2.1	8.2	8.6	22.3	28.1	2.4	2.1	2.1	1.5	2.2
Pittsburgh	2.4	3.9	7.7	15.9	65.0	1.5	2.3	3.4	1.9	2.7
Buffalo	2.7	3.9	8.1	15.4	22.8	1.9	1.9	2.3	2.4	5.0
Camden	4.4	5.9	4.9	4.5	4.0	4.2	6.8‡	5.1‡	0.8‡	5.1‡

* Data for 1925 only.

† Typhoid deaths for Scranton furnished by Pennsylvania Department of Health, Harrisburg.

‡ Previous figure revised on basis of 1930 population.

good record in 1926–1928, brings up at the foot of the group in 1930; the relatively high rate in that city suggests that a carrier may be at work somewhere in the vicinity. Camden for the third year in succession reports a typhoid rate that should prompt inquiry. The Camden five year average for 1926–1930 is four times that of Philadelphia.

Eight of the nine cities of the South Atlantic group had higher rates in 1930 than in 1929 (table 3), with considerable increases in Norfolk

TABLE 3

Death rates of cities in South Atlantic states from typhoid per hundred thousand population

	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910	1930	1929	1928	1927	1926
Richmond	1.9	5.7	9.7	15.7	34.0	2.2	1.5	3.1	0.0	2.6
Norfolk	2.2	2.8	8.8	21.7	42 1	3.8	0.8*	2.4*	1.6*	2.4*
Washington	2.8	5.4	9.5	17.2	36.7	3.3	2.7*	3.1*	2.1*	2.8*
Baltimore	3.2	*4.0	11.8	23.7	35.1	3.3	2.6	3.8	1.8	4.7
Miami	3.5					1.8	1.4	1.4	4.6	8.5
Wilmington	3.7	4.7	25.8†	23.2‡	33.0	4.7	0.9*	6.4*	3.6*	2.8*
Tampa	3.8	19.1	43.9§			3.0	0.9	7.1	6.5	23.5
Jacksonville	4.4					0.0	0.8*	2.4*	8.4*	10.5*
Atlanta	11.1	14.5	14.2	31.4	58.4	10.3	6.5	7.4	14.0	17.2

* Previous figure revised on basis of 1930 population.
† Data for 1916 only.
‡ Lacks data for 1913.
§ Lacks data for 1916.

TABLE 4

Death rates of cities in East North Central states from typhoid per hundred thousand population

	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910	1930	1929	1928	1927	1926
South Bend						0.0				
Peoria	0.2	3.7	5.7	16.4	15.7*	0.0	0.0	0.0	1.0	0.0
Chicago	0.6	1.4	2.4	8.2	15.8	0.6	0.6	0.5	0.7	0.8
Milwaukee	0.8	1.6	6.5	13.6	27.0	0.3	0.4	0.7	0.9	1.7
Cleveland	1.0	2.0	4.0	10.0	15.7	1.5	0.5	0.6	1.0	1.4
Grand Rapids	1.0	1.9	9.1	25.5	29.7	0.6	0.6	0.6	1.9	1.3
Youngstown	1.1	7.2	19.2	29.5	35.1	2.3	1.0	0.0	2.4	0.0
Detroit	1.3	4.1	8.1	15.4	22.8	1.1	0.9	1.0	1.2	2.2
Canton	1.4	3.3	8.9			1.9	0.8	0.8	0.0	3.6
Akron	1.5	2.4	10.6	21.0	27.7†	1.2	1.7	0.4	2.8	1.5
Flint	1.6	4.6	22.7	18.8	46.9	0.6	1.3	3.4	2.1	0.7
Dayton	1.9	3.3	9.3	14.8	22.5	0.5	1.6	2.7	1.7	2.3
Columbus	2.1	3.5	7.1	15.8	40.0	2.7	2.3	1.7	2.0	1.7
Cincinnati	2.5	3.2	3.4	7.8	30.1	2.4	1.9	1.7	3.9	2.7
Indianapolis	2.7	4.6	10.3	20.5	30.4	1.9	1.5	3.6	1.3	5.4
Toledo	3.0	5.8	10.6	31.4	37.5	2.1	0.3	4.8	2.9	4.7
Fort Wayne	4.2	12.9	7.3			2.6	5.3	5.5	1.9	5.8
Evansville	6.2	5.0	17.5	32.0	35.0	6.8	4.9	3.0	7.1	9.4

* Lacks data for 1907, 1908 and first four months of 1909.
† Data for 1909 and 1910 only.

and Wilmington. In spite of this rise, however, the 1926–1930 averages are all lower than the available averages for 1921–1925, the reductions in Richmond, Washington and Tampa being especially noteworthy. The two newcomers in this group, Miami and Tampa,

TABLE 5

Death rates of cities in East South Central states from typhoid per hundred thousand population

	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910	1930	1929	1928	1927	1926
Louisville	3.7	4.9	9.7	19.7	52.7	1.6	3.0*	3.1*	3.5*	7.5*
Birmingham	8.0	10.8	31.5	41.3	41.7	4.6	7.0	7.2	12.9	8.5
Chattanooga	8.0	18.6	27.2	35.8‡		0.8	6.2	6.5	13.9	12.6
Memphis	9.3	18.9	27.7	42.5	35.3	4.7	4.9*	9.5*	11.5*	15.7*
Knoxville	10.7	20.8	25.3†			1.9	11.7	12.0	10.2	17.9
Nashville	18.2	17.8	20.7	40.2	61.2	12.3	12.8	15.0	16.0	35.0

* Previous figure revised on basis of 1930 population.

† Data for 1920 only.

‡ Data for 1914 and 1915 only.

TABLE 6

Death rates of cities in West North Central states from typhoid per hundred thousand population

	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910	1930	1929	1928	1927	1926
Minneapolis	0.8	1.9	5.0	10.6	32.1	1.3	0.6	0.2	0.7	1.4
Duluth	1.1	1.7	4.4	19.8	45.5	2.0	0.0	0.0	2.7	0.8
Wichita	1.2	6.3				1.8	0.0	2.0	2.1	0.0
Omaha	1.3	3.3	5.7	14.9	40.7	0.9	0.4	2.7	0.9	1.8
St. Paul	1.4	3.4	3.1	9.2	12.8	0.7	1.0	1.3	2.8	1.2
Kansas City, Kan.	1.7	5.0	9.4	31.1	74.5*	0.0	2.5	1.7	0.0	4.3
St. Louis	2.1	3.9	6.5	12.1	14.7	2.5	1.5	2.2	1.9	2.3
Des Moines	2.4	2.2	6.4	15.9	23.7	1.4	3.2	1.3	3.3	2.7
Kansas City, Mo.	2.8	5.7	10.6	16.2	35.6	2.2	3.2	2.3	2.9	3.5

* Lacks data for 1906 and 1907.

report good records for 1930. Tampa seems to have had an appalling amount of typhoid in 1916–1920 and even as late as 1926.

Jacksonville, reporting no typhoid deaths in 1930, gets a place on the Honor Roll. Atlanta records 1926–1930 as its best five year period.

The cities of the East North Central group (table 12) average a little more typhoid in 1930 (1.14) than in 1929 (0.96). Such cities

TABLE 7

Death rates of cities in West South Central states from typhoid per hundred thousand population

	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910	1930	1929	1928	1927	1926
San Antonio	4.6	9.3	23.3	29.5	35.9	3.9	3.1	2.3	5.7	7.8
Houston	4.8	7.6	14.2	38.1	49.5*	3.8	2.8	6.0	5.6	6.0
Fort Worth	5.9	6.1	16.3†	11.9	27.8	4.9	2.8	7.0	4.3	10.7
Dallas	7.3	11.2	17.2			6.5	9.4	5.0	6.6	8.9
Oklahoma City	7.4‡					7.0	6.2	10.0	6.4	
Tulsa	8.3	16.2§				2.1	3.0¶	4.7¶	10.7¶	16.8¶
El Paso	9.1	10.8	30.7	42.8		6.8	6.0¶	12.3¶	9.5¶	10.8¶
New Orleans	9.9	11.6	17.5	20.9	35.6	6.5	9.1	7.2	8.0	18.6

* Data for 1910 only.
† Lacks data for 1918 and 1919.
‡ Lacks data for 1926.
§ Lacks data for 1921 and 1922.
¶ Previous figure revised on basis of 1930 population.

TABLE 8

Death rates of cities in Mountain and Pacific states from typhoid per hundred thousand population

	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910	1930	1929	1928	1927	1926
San Diego	1.0	1.6	7.9	17.0	10.8	0.0	1.6	1.7	0.9	0.9
Long Beach	1.1	2.1*				0.7	0.6	2.1	1.4	0.7
Oakland	1.2	2.0	3.8	8.7	21.5	1.0	1.1	1.1	2.2	0.8
Los Angeles	1.5	3.0	3.6	10.7	19.0	1.5	1.5†	1.9†	1.2†	1.3†
Tacoma	1.8	3.7	2.9	10.4	19.0	0.9	1.7	0.0	2.8	3.8
Salt Lake City	1.9	6.0	9.3	13.2	41.1	2.8	0.7	2.9	2.2	0.7
San Francisco	2.0	2.8	4.6	13.6	26.3	1.3	1.0	3.4	1.7	2.5
Seattle	2.2	2.6	2.9	5.7	25.2	2.2	2.3	1.3	2.7	2.3
Spokane	2.2	4.4	4.9	17.1	50.3	1.7	0.0	1.8	0.9	6.5
Portland	2.3	3.5	4.5	10.8	23.2	1.0	2.7†	2.4†	2.4†	2.9†
Denver	2.6	5.1	5.8	12.0	37.5	2.1	2.0	2.7	2.8	3.5

* Lacks data for 1921.
† Previous figure revised on basis of 1930 population.

as Cleveland, Cincinnati and Toledo (table 4) record substantial increases, and others like Chicago, Detroit and Grand Rapids show no

TABLE 9

*Death rates from typhoid in 1930**

Honor Roll: No typhoid deaths

Bridgeport	Peoria
Jacksonville	South Bend
Kansas City, Kan.	San Diego
New Haven	Utica
Paterson	Worcester

First Rank (from 0.1 to 1.9 Deaths per Hundred Thousand)

Newark	0.2	Oakland	1.0†
Jersey City	0.3	Portland	1.0
Milwaukee	0.3	Waterbury	1.0
Dayton	0.5	Detroit	1.1
Syracuse	0.5†	Akron	1.2‡
Chicago	0.6	Minneapolis	1.3‡
Flint	0.6†	San Francisco	1.3
Grand Rapids	0.6†	Des Moines	1.4
Long Beach	0.7	Scranton	1.4
St. Paul	0.7	Cleveland	1.5
Springfield	0.7†	Los Angeles	1.5‡
Yonkers	0.7	Pittsburgh	1.5‡
Albany	0.8	Louisville	1.6
Boston	0.8	Spokane	1.7‡
Chattanooga	0.8	Miami	1.8
Cambridge	0.9	Rochester	1.8
Erie	0.9	Wichita	1.8
New York	0.9	Buffalo	1.9
Omaha	0.9	Canton	1.9
Philadelphia	0.9	Indianapolis	1.9
Reading	0.9	Knoxville	1.9
Tacoma	0.9	Somerville	1.9
Lynn	1.0		

Second Rank (from 2.0 to 4.9)

Duluth	2.0‡	Toledo	2.1
Providence	2.0‡	Tulsa	2.1
Denver	2.1	Kansas City, Mo.	2.2‡

* Note that the ranks have been arranged differently from those in the corresponding tables of previous reports. Formerly the Honor Roll included all cities with rates from 0.0 to 1.9. The new arrangement puts into Second Rank the cities that used to be called First Rank, and the new Third and Fourth Ranks correspond to the old Second and Third.

† The single typhoid death reported was stated to be in a nonresident.

‡ One-third or more of the reported typhoid deaths were stated to be in nonresidents.

TABLE 9—*Concluded*

Second Rank (from 2.0 to 4.9)—*Concluded*			
Richmond	2.2	Tampa	3.0‡
Seattle	2.2‡	Baltimore	3.3‡
Youngstown	2.3‡	Washington	3.3
Cincinnati	2.4	Houston	3.8‡
Hartford	2.4‡	Norfolk	3.8
Trenton	2.4‡	San Antonio	3.9
St. Louis	2.5	Camden	4.2‡
Fall River	2.6	Elizabeth	4.4‡
Fort Wayne	2.6	Birmingham	4.6‡
Columbus	2.7‡	Memphis	4.7‡
New Bedford	2.7	Wilmington	4.7
Salt Lake City	2.8‡	Fort Worth	4.9‡
Third Rank (from 5.0 to 9.9)			
Lowell	5.0	El Paso	6.8
Dallas	6.5‡	Evansville	6.8
New Orleans	0.5	Oklahoma City	7.0
Fourth Rank (10.0 and Over)			
Atlanta	10.3‡	Nashville	12.3‡

improvement. Only five of the seventeen cities reporting in 1929 record any decline in typhoid in 1930. On the other hand, sixteen of the seventeen have lower five year averages for 1926–1930 than for 1921–1925. The single exception, Evansville, had a relatively high rate (6.8) in 1930 and the highest five year average (6.2) in the group. What is the matter with Evansville? In 1916–1920 and again in 1921–1925 it had about the same typhoid average as Youngstown, but in 1926–1930 the Youngstown average was 1.1 and that for Evansville 6.2. Among the newcomers in this group Peoria seems to have a remarkable history, only one typhoid death being reported in seven years, a record apparently unequaled in the country.

The East South Central cities (table 5) all show improvement in their 1930 typhoid rates compared with those for 1929. The two newcomers, Chattanooga and Knoxville, show the greatest reductions and have among the lowest rates in the group (0.8 and 1.9). Knoxville, whose high typhoid rate in the past has several times been made the subject of special investigation, shows marked improvement in its five year average. Memphis, which has given steadily improving rates since 1926, reaches in 1930 the relatively low figure of 4.7

and for the first time has a five year average under 10. Birmingham also in 1930 breaks its previous low record, and Louisville gives the excellent typhoid rate of 1.6 for 1930 and 3.7 for 1926–1930. The cities of this group, most of which have had to struggle with untoward conditions, have made perhaps the greatest advance in the past decade of any group in the country. Nashville is apparently having the greatest difficulty of any of these cities in bringing its typhoid rate down. It is the only one of the group to report a higher average in 1926–1930 than in 1921–1925. This is largely due, however, to excessive typhoid in 1926; since that year there has been steady, if not marked, improvement. What Memphis, Birmingham

TABLE 10

Number of cities with various typhoid death rates

	NUMBER OF CITIES	10.0 AND OVER	5.0 TO 9.9	2.0 TO 4.9	1.0 TO 1.9	0.1 TO 0.9	0.0
1906–1910	77	75	2	0	0	0	0
1911–1915	79	58	19	2	0	0	0
1916–1920	84	22	32	30	0	0	0
1921–1925	89	12	17	48	12	0	0
1926–1930	92	3	10	30	37	12	0
1926	91	9	13	28	20	15	6
1927	92	6	10	28	27	13	8
1928	92	5	9	29	22	17	10
1929	92	2	9	21	27	25	8
1930	93	2	6	30	23	22	10

and Louisville have done, Nashville should be able to do. The city officials of Nashville should compare the 1916–1920 typhoid averages for this group with those of 1926–1930 and note that Nashville is the only one of the six cities that has failed to achieve a marked reduction in this preventable disease.

Several cities in the West North Central group (table 6), such as Minneapolis and St. Louis, had more typhoid deaths in 1930 than for some years previously, and the group rate as a whole (table 12) shows a slight increase (1.56 to 1.74) similar to that of the East North Central states. Some sanitarians will be inclined to relate the typhoid increase in these two regional groups to the hot weather and drought of the summer of 1930. If such a connection exists, it still remains to determine whether the effect of weather conditions lies in

an influence on individual susceptibility or on channels of transmission. All the cities of this group except Des Moines make a better record for 1926–1930 than for 1921–1925.

The West South Central cities (table 7) have long had relatively unsatisfactory typhoid rates but appear to be gradually pulling out

TABLE 11

*Total typhoid rate for seventy-eight cities, 1910–1930**

	POPULATION	TYPHOID DEATHS	TYPHOID DEATH RATE PER 100,000
1910	22,573,435	4,637	20.54
1911	23,211,341	3,950	17.02
1912	23,835,399	3,132	13.14
1913	24,457,989	3,285	13.43
1914	25,091,112	2,781	11.08
1915	25,713,346	2,434	9.47
1916	26,257,550	2,191	8.34
1917	26,865,408	2,016	7.50
1918	27,086,696†	1,824†	6.73
1919	27,735,083†	1,151†	4.15
1920	28,244,878	1,088	3.85
1921	28,859,062	1,141	3.95
1922	29,473,246	963	3.26
1923	30,087,430	950	3.16
1924	30,701,614	943	3.07
1925	31,315,598	1,079	3.44
1926	31,929,782	907	2.84
1927	32,543,966	648	1.99
1928	33,158,150	628	1.89
1929	33,772,334	537	1.59
1930‡	34,386,717	554	1.61

* The following fifteen cities are omitted from this table because data for the full period are not available: Canton, Chattanooga, Dallas, Fort Wayne, Jacksonville, Knoxville, Long Beach, Miami, Oklahoma City, South Bend, Tampa, Tulsa, Utica, Wichita, Wilmington.

† Data for Fort Worth lacking.

‡ The rate for the ninety-three cities in 1930 is 1.64 (total population, 36,326,335; typhoid deaths, 605).

of their deep rut. Dallas fared much better in 1930 than in 1929. New Orleans breaks its previous low record, bringing its five year average for the first time under 10. Tulsa leads the group for 1930 by a good margin and, while its five year average is high, the next period seems likely to tell a different story. All the cities in this

group had less typhoid in 1926–1930 than in the previous quinquennium.

The group of Mountain and Pacific cities (table 8) again makes a good showing in 1930, Salt Lake City being the only one to rise materially above its 1926–1930 average. The group as a whole (table 12) manifests about the same proportional 1929–1930 reduction (1.51 to 1.43) as that shown by the cities in New England and the Middle Atlantic states. In every instance the five year average for 1926–1930 is lower than that for 1921–1925, the improvement being marked in several cities (Los Angeles, Long Beach, Tacoma,

TABLE 12

Total typhoid death rate per hundred thousand population for ninety-three cities according to geographic divisions

	1930 POPULATION	1930 TYPHOID DEATHS	TYPHOID DEATH RATES			
			1930	1929	1926–1930	1925
New England	2,601,824	32	1.23	1.35	1.31	2.48
Middle Atlantic	12,650,337	131	1.03	1.19	1.40	2.97
South Atlantic	2,322,692	90	3.87	2.68	4.50	7.01*
East North Central	9,302,752	106	1.14	0.96†	1.29†	2.32†
East South Central	1,200,032	51	4.25	6.71	8.31	13.00
West North Central	2,648,663	46	1.74	1.56	1.83	3.43
West South Central	1,835,646	98	5.34	5.87	7.32‡	13.08§
Mountain and Pacific	3,764,389	54	1.43	1.51	1.80	2.33

* Data for Jacksonville and Miami lacking.
† Data for South Bend for 1925–1929 lacking.
‡ Data for Oklahoma City in 1926 lacking.
§ Data for Oklahoma City lacking.

Salt Lake City, Spokane and Denver). San Diego gets a place on the Honor Roll.

Ten of the ninety-three cities reported that no typhoid deaths occurred within their limits in 1930 (table 9). This is the largest number, although not the largest proportion, of cities with perfectly clear records ever reported in these articles. No city has yet had five successive years without a typhoid death, although New Haven, Yonkers, and Peoria have come close to it. The five year averages make a particularly interesting comparison now that annual rates are in many localities so low. Of the eighty-nine cities in this summary for which both 1921–1925 and 1926–1930 averages are available

(these data are lacking for Jacksonville, Miami, Oklahoma City and South Bend), only five (Des Moines, Evansville, Lowell, Nashville and Waterbury) failed to show improvement in 1926–1930 as compared with 1921–1925. It is interesting to note also that in 1921–1925 no city had a typhoid average below 1.0 (table 10), while in 1926–1930 there were twelve cities with an annual average of less than one typhoid death per hundred thousand inhabitants. Furthermore, in 1921–1925 there were only twelve cities with an average less than 2.0; in 1926–1930 there were forty-nine.

The total typhoid mortality rate in 1930 in the seventy-eight cities for which records are available beginning with 1910 (table 11) was almost exactly the same (1.61) as that for 1929 (1.59). Similar halts in the steady typhoid diminution have been noticed three other times in the past twenty years, in 1913, 1921 and 1925. After each pause the downward progress was resumed. There is no reason to suppose that typhoid reduction in the large cities of the United States has been permanently checked.

ABSTRACTS OF WATER WORKS LITERATURE[1]

FRANK HANNAN

Key: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

Purification Experiments on Raw Water. FRITZ DILLER. Wochbl. Papierfabr., 60: 831–2; Papier-Fabr., 27: 521–3, 1929. From Chem. Abst., 23: 5524, November 20, 1929. Data are given on the purification of bog water with aluminum sulfate (I), with lime and alum (II), and with sulfuric acid and alum with pH control (III). I was satisfactory except that it failed to remove manganese. II removed manganese, but required careful control both in dosage and in acidifying after filtration to improve the color. III gave more rapid coagulation than either I or II (20 minutes *vs.* 3½ hours), but again failed to remove manganese. This can be removed by filtration through a layer of manganous pyrites, which is expensive. The pH (5.5) giving best purification in III was too acid for complete removal of manganese in this way.—*R. E. Thompson.*

The Treatment of Waste Waters from Sugar-Refineries. JOSEF JASKÓLSKI. Listy Cukrovar., 48: 1–7, 9–14, 1929. From Chem. Abst., 23: 5606, November 20, 1929. Several processes are reviewed. Chlorine passed into an absorption tower into which the waste water was sprayed functioned as sterilizing and deodorizing agent. The presence of chlorine in the return water aids sedimentation, 55 percent of the usual lime dosage giving good sedimentation. This is due to the fact that the chlorine prevents fermentation; during fermentation protective colloids form, preventing flocculation and sedimentation of organic matter. Optimum results were obtained when the pH exceeded 9. —*R. E. Thompson.*

Purification of Tannery Effluents by Colloidal Clay. J. BESSE. J. Intern. Soc. Leather Trades Chem., 13: 503–7, 1929. From Chem. Abst., 23: 5608, November 20, 1929. By acidifying with sulfuric acid and treating with suspension of clay, practically all impurities are precipitated, effluent being clear, free from sulfide, low in nitrogen, and non-foaming.—*R. E. Thompson.*

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[1] Vacancies on the abstracting staff occur from time to time. Members desirous of coöperating in this work are earnestly requested to communicate with the chief abstractor, Frank Hannan, 285 Willow Avenue, Toronto 8, Ontario, Canada.

The Purification of Tannery Effluents by Argillaceous Colloids. J. BESSE. Cuir tech., 18: 244–6, 1929. From Chem. Abst., 23: 5608, November 20, 1929. Lime liquors were made slightly acid to phenolphthalein and equal volume of 2 percent brick clay suspension, prepared by agitating clay in 0.05 percent sodium carbonate, was added. Analyses of 4 lime liquors showed that in all cases the total nitrogen, sulfides, and solids of the solutions after flocculation were 10 percent, or less, of the original concentrations. The great disadvantage is the large volume of water required.—*R. E. Thompson.*

Disposal of Creamery Wastes. CLARE E. SLAUGHTER. Mich. State Coll., Eng. Expt. Sta. Bull., 18: 27 pp., 1928. From Chem. Abst., 24: 169, January 10, 1930. High acidity inhibits reduction of the wastes. The acidity increases as the oxygen consumed value is reduced. The acidity in a septic tank with 1-day retention is about 400 p.p.m., and with 3-day retention, about 600 p.p.m. The corresponding oxygen consumed values are 400 and 250 p.p.m., respectively. Sludge does not form readily and even with 3-day retention the effluent is extremely offensive and contains large amounts of organic matter. A contact bed reduced the oxygen consumed and increased the acidity about 7 percent. Septic action is much facilitated by addition of lime to pH 7.6. The effluent so obtained, low in acidity and suspended solids, is suitable for sand filtration. The method is applicable to any septic tank, increasing its efficiency and reducing sludge, scum, and odor. Chemical precipitation with 0.5 gram ferrous sulfate ($FeSO_4 \cdot 7\ H_2O$) per liter followed by adjustment to pH 7.8 with lime resulted in settlement in 10 minutes and removal of most of the bacteria and solids. The effluent is clear and inoffensive, the relative stability being 60 percent, and may be further purified by secondary storage or oxidation on a sand filter. Such treatment at from 3 to 4 cents per 1000 gallons is feasible for small creameries, the waste being treated in as concentrated a form as possible.—*R. E. Thompson.*

Measurement of Fluids in Pipe Lines. II. Method of Velocity Change. H. ESCHER. Commonwealth Eng., 17: 53–60, 1929; cf. C.A., 23: 4981. From Chem. Abst., 24: 174, January 10, 1930.—*R. E. Thompson.*

New Thoughts in the Theory of Flowing. J. MAERCKS. Glückauf, 65: 1234–41, 1929. From Chem. Abst., 24: 174, January 10, 1930. According to the new theory the frictional resistance is a function of the REYNOLD number for any kind of flowing. This number is explained and applied to flowings in pipes, etc.—*R. E. Thompson.*

Potable Water Supply of the Low Countries. KRUL and LIEFRINCK. Tech. sanit. munic., 24: 155–60, 1929. From Chem. Abst., 24: 178, January 10, 1930. The water consumption per capita varies from 48 liters per capita at Haarlem to 114 liters at The Hague. The various filtration and purification methods are briefly described.—*R. E. Thompson.*

Subterranean Waters on the Territory of the City of Zagreb (Jugoslavia). H. IVEKOVIČ. Arhiv. hem. i farm., 3: 113–7 (117 German), 1929. From

Chem. Abst., **24**: 178, January 10, 1930. On the basis of numerous analyses a relationship between the composition of water and its geographical position is found and a new theory is set up against the assertion of other authors concerning the origin of underground water.—*R. E. Thompson.*

Limnological Methods. CHANCEY JUDAY. Arch. Hydrobiol., **20**: 517–24, 1929. From Chem. Abst., **24**: 178, January 10, 1930. The field and laboratory methods used by JUDAY and co-workers in the chemical analysis of lake water are discussed.—*R. E. Thompson.*

A New Apparatus for Water Sterilization by Hypochlorites. A. SIVADE. Tech. sanit. munic., **24**: 136–40, 1929. From Chem. Abst., **24**: 179, January 10, 1930. Description of apparatus.—*R. E. Thompson.*

Boiler Feed Water Treatment. DAVID BROWNLIE. J. Inst. Brewing, **35**: 410–14, 1929. From Chem. Abst., **24**: 180, January 10, 1930.—*R. E. Thompson.*

Modern Boiler Water Treatment. K. HOFER. Glückauf, **65**: 1067, 1928. From Chem. Abst., **24**: 180, January 10, 1930. A description and discussion of the different methods of boiler water treatment.—*R. E. Thompson.*

The Chemical Control of the Preparation of Water for Boilers. V. SÁZAVSKÝ. Listy Cukrovar, **48**: 41–51, 1929. From Chem. Abst., **24**: 180, January 10, 1930. A detailed review of suitable analytical methods.—*R. E. Thompson.*

Boiler Metal Cracking. MAX HECHT and D. S. MCKINNEY. Power, **70**: 633–6, 1929. From Chem. Abst., **24**: 181, January 10, 1930. Cracking of a mud-drum blow-off pad was attributed to the combined effects of corrosion and strain.—*R. E. Thompson.*

Industrial Water Treatment. I. Filtration and Lime Soda Plants. GEORGE CAWLEY. Power, **70**: 642–5, 1929. From Chem. Abst., **24**: 180, January 10, 1930. Descriptions and sectional elevations are given of a pressure sand filter, a cold continuous lime-soda softener, and a hot process softener.—*R. E. Thompson.*

Tests of the Effect of Sulfate Soils and Waters on Concrete. RAYMOND WILSON and ALBERT CLEVE. Portland Cement Assoc., Rept. of Director of Research 1928, 70–4. From Chem. Abst., **24**: 217, January 10, 1930. Results of field tests. Among workable concretes there was found a close relationship between 28-day compressive strength and resistance to sulfate attack. The only concretes (without surface coatings) which were in good condition after 7 years exposure at one site were those made with 5 gallons of water, or less, per sack of cement. Harsh working mixtures, lean mixtures, and very wet mixtures showed poor resistance to attack. Integral water- and alkali-proofing compounds imparted no additional resistance. Concrete coated with linseed oil (3 coats) was in perfect condition after 7 years' exposure. Some bituminous coatings gave good protection, if good adhesion to the concrete was obtained.—*R. E. Thompson.*

Accelerated Biological Filtration in the Waterworks Industry. CHR. WICHERS. Pharm. Weekblad, **66**: 694–6, 1929. From Chem. Abst., **24**: 178, January 10, 1930. Purification of municipal water supply by filtration through sand is usually performed at the rate of 100 liters per hour per square meter of filtering surface. The system at Groningen treats well water in Haren and surface water in de Punt. An experimental plant was erected at de Punt to determine whether the rate of filtration can safely be increased. Five open filters were constructed, each containing a 100-centimeter layer of sand over gravel of various degrees of fineness. They were operated to deliver 50, 100, 200, 300 and 400 liters water per square meter per hour and tested with water of varying states of purity. Chemical and biological examinations were made of the filtered water and of the flora present in the residues. It was found that sand filtration operates through mechanical, physical, chemical, and biological processes. The upper layer retains suspended matter, but this covering is not necessary for the bacteriological processes. Biological factors play a rôle in the destruction of organic matter by oxidation. If the ratio of organic matter to oxygen is too high, filtration is not efficient. Bacteria are destroyed by protozoa, numerous diatoms and protozoa being found in the upper layers. No slime coatings were observed on the sand grains. The filters operated satisfactorily when the rate of filtration was increased to 300–400 liters per square meter per hour.—*R. E. Thompson.*

Removing Phenols from Waste Waters of Gasworks, etc. Posseyer-Meguin A.-G. Ger. 481,738, August 27, 1922. From Chem. Abst., **24**: 229, January 10, 1930. The waters are treated with vapors of suitable absorbents for phenol, e.g., benzol, benzine, or trichlorethylene. The waters may be passed down a packed tower in the same direction as the vapors or in counter current thereto. An alternative method consists in extracting the water with liquid trichlorethylene.—*R. E. Thompson.*

Applied Research on Corrosion for Steam Power Plants. HANS BALCKE. Brennstoff- u. Wärmewirtschaft, 11: 287–91, 345–8, 1929. From Chem. Abst., **24**: 327, January 20, 1930. Different types of machinery used for prevention of corrosion are described.—*R. E. Thompson.*

Studies on the Treatment of Milk-Products Waste. Progress Rept. No. 1. E. F. ELDRIDGE. Mich. State Coll. Eng. Expt. Sta., Bull. No. **24**: 3–26, 1929. From Chem. Abst., **24**: 188, January 10, 1930. The use of ferrous sulfate and lime for chemical precipitation, if properly carried out, gives a constantly clarified effluent in good condition for secondary treatment on a biological filter. Four pounds of copperas per 1000 gallons is minimum quantity for proper and efficient treatment. Lime or sodium hydroxide sufficient to give phenolphthalein alkalinity is necessary for good floc formation. By adding copperas first, less lime is needed for floc formation, provided waste is fresh. If waste is sour, the reverse order of application is necessary. Complete removal of oxygen-consuming substances does not seem possible. In biological treatment, the indications are that between 70 and 80 percent of the oxidation takes place in upper 4 feet of filter. The reduction depends largely

on strength of raw waste. Rate of application of waste to filter depends on degree of stability desired in effluent. Indications are that each square foot of filter will handle daily a quantity of waste requiring approximately 43 grams of oxygen based on the oxygen consumed determinations.—*R. E. Thompson.*

New Reagent for the Colorimetric Determinations of Minute Amounts of Copper. THOMAS CALLAN and J. A. RUSSELL HENDERSON. Analyst, **54**: 650–3, 1929. From Chem. Abst., **24**: 312, January 20, 1930. Sodium diethyldithiocarbamate gives a brown precipitate with copper ions; more sensitive than potassium ferrocyanide or sodium ethylxanthate test. The color is easily matched with standards and serves to detect 1 part of copper in 100 million parts of distilled water. The color is the same in alkaline solutions, but cyanides inhibit the reaction by forming complex anion with the copper. To carry out the test, make the copper solution ammoniacal, add 10 cc. of 0.1 percent solution of the reagent in water, dilute to 100 cc., and mix.—*R. E. Thompson.*

Industrial Water Treatment. II. Zeolite Softeners. GEORGE CAWLEY. Power, **70**: 716–8, 1929; cf. C.A., **24**: 180. From Chem. Abst., **24**: 448, January 20, 1930. The principles of zeolite softening are explained.—*R. E. Thompson.*

Antiseptic Value of Chlorine with Reference to Animal Skin Disinfection. R. C. HOWARD, G. E. ROCKWELL and W. L. CRIST. J. Am. Leather Chem. Assocn., **24**: 456–72, 1929. From Chem. Abst., **24**: 261, January 10, 1930. A study was made of the concentration and time required for disinfection of solutions inoculated with hay bacillus spores. Rate of sterilization decreases with increasing number of organisms present and in presence of sodium chloride, organic matter, and ammonia (except in presence of organic matter). Treatment lasting 1 hour was necessary to sterilize solution containing 24 millions of bacteria per cubic centimeter, and 3 hours for 120 millions, using 120 p.p.m. of chlorine. Longer time was required for the older spores. By increasing chlorine concentration sufficiently, it is possible to kill all organisms in liquor surrounding a skin, but if the skin is then transferred to a broth, growth always occurs, even when chlorine concentration has been so high that the skins are damaged (1500 p.p.m.). Skins rapidly remove chlorine from solution. Addition of ammonia retards chlorination of skins and promotes sterilization of the soak water. Tannery effluents consume chlorine in proportion to their nitrogen and bacterial contents. Chlorine should be added to give a definite concentration after 15 minutes or 1 hour.—*R. E. Thompson.*

Microchemical Determination of Iodine in Natural Substances. J. F. REITH. Pharm. Weekblad, **66**: 829–45, 1929; cf. C.A., **23**: 2122, 2675. From Chem. Abst., **24**: 313. January 20, 1930. A study of various methods for determining small quantities of iodine and of disturbing influence of certain anions such as chlorine, bromine, nitrite, nitrate, sulfate, chlorate, and bromate in titrations with sodium thiosulfate and starch.—*R. E. Thompson.*

Conversion of Sodium Carbonate to Caustic Soda Within Steam Boilers. C. E. JOOS. Power, **70**: 762–3, 1929. From Chem. Abst., **24**: 448, January

20, 1930. Under temperature and pressure conditions within a boiler, one-half to two-thirds of the carbonate will be converted to hydroxide.—*R. E. Thompson.*

The Influence of Impurities in Oxygen and Acetylene upon Welding. W. RIMARSKI. Z. Ver. deut. Ing., 73: 1467–8, 1929; cf. C.A., 23: 4344. From Chem. Abst., 24: 321, January 20, 1930. Comparative experiments with 99.5, 98.7, 98.1, and 97.5 per cent oxygen used with acetylene showed greater speeds at higher concentrations at all pressures employed (1 to 4 kgm./sq. cm.), the increase being greatest at lowest concentrations. At the same prices for oxygen of 99.5 and 98.0 percent content, the richer oxygen is most economical. By preheating the oxygen the time of cutting, etc., is decreased; by this means the influence of high nitrogen in the oxygen may be diminished. The economy effected must be determined in each case. Nitrogen in the oxygen and water in the acetylene increased the time of melting. The skill of the operator is of greater importance than the factors mentioned above. A higher water content than 2 percent in the acetylene is to be avoided, as it causes considerable loss of efficiency.—*R. E. Thompson.*

The Purification of Waste from Coal Mines and Coal Washings. A. REICH. Gesundh.-Ing., 52: 777, 1929. From Chem. Abst., 24: 486, January 20, 1930. Present methods of waste treatment, including electrolysis, use of Emscher tanks, and the TRAVIS system, are outlined. Free acid is often present in wastes from coal mines. Fish life is usually impossible in the unpurified waste.—*R. E. Thompson.*

Investigation by Means of Diffusion Studies of Protection Against Fire and Explosions in Tunnels. ARTHUR RINGEL. Gesundh.-Ing., 52: 759, 1929. From Chem. Abst., 24: 501, January 20, 1930. Analyses of gas from tunnels were made. Explosion zones with different types and ratios of gases are shown. The poisonous and explosive concentrations of carbon dioxide, carbon monoxide, hydrogen sulfide, chlorine, nitric oxide, hydrogen, acetylene, benzol, and methane are given. Studies were made of the rates of diffusion of the gases in tunnels.—*R. E. Thompson.*

Corrosion of Underground Steel Structures and Its Prevention. ROBERT E. WILSON. J. West. Soc. Eng., 34: 578–95, 1929. From Chem. Abst., 24: 326, January 20, 1930. The phenomena which occur when iron is in contact with various chemical combinations and the theories regarding the same are discussed and the means found most effective for protecting oil pipe lines are described. Special attention is paid to corrosion of the outside of pipe by the soil. Three essential factors are involved: the kind of pipe, the nature of the soil, and the protective coating used. Causes of coating failures are: chemical or physical, due to changes in the coating material itself; and mechanical, due to improper handling or other causes not directly connected with the coating. An unbroken coating is important. Wrapping of the coated pipe with asbestos paper impregnated with asphalt and applying a second coating of asphalt to protect the fabric and give a smooth surface apparently gives the best results.—*R. E. Thompson.*

Soil Corrosion Studies, 1927–28. K. H. LOGAN. Bur. Standards J. Research, 3: 275–302, 1929. From Chem. Abst., 24: 327, January 20, 1930. Bureau of Standards soil corrosion test specimens (C.A., 23: 2922) removed in 1928 were weighed to determine the corrosion and pitting. Supplementary field investigations have corroborated the results, indicating that the rate of penetration of pits in cast iron decreases with the age of the pipe. In oil lines, part of the pitting is attributed to galvanic action due to concentration effects when a pipe passes through different soils. The laboratory tests check fairly well with those of 1926, indicating that the conditions of the test have reached a nearly steady state. There are no data showing that the tentative conclusions in the earlier reports should be changed. On the average, corrosion and pitting rates decrease, pitting progressing more slowly than overall corrosion. No final conclusions are to be drawn regarding the ultimate life of any material.—*R. E. Thompson.*

Applied Research on Corrosion for Steam Power Plants. HANS BALCKE. Brennstoff- u. Wärmewirtschaft, 11: 287–91, 345–8, 1929. From Chem. Abst., 24: 327, January 20, 1930. Different types of machinery used for prevention of corrosion are described.—*R. E. Thompson.*

Chlorinated Drinking Water: The Use of Activated Charcoal Filters. KARL IMHOFF and F. SIERP. Surveyor, 76: 225, September 6, 1929. Pure phenol does not impart a taste to water in concentrations as high as 1 in 1 million, but when chlorine is also present, taste can sometimes be detected in a dilution of 1 in 500 million. There are a number of sources of phenol, such as coke and gas works, charcoal works, lignite distillation plants, protective coatings on mains and tanks, decomposing organic river mud, etc. In addition to phenol there are a number of other substances arising from similar sources which cause taste, e.g., cresol, xylenol, anisol, guaiacol, anthracene, naphthalene, etc. Tar from charcoal works, owing to its solubility in water, is exceptionally troublesome. It collects on the bottom of the stream or on the bank and gives off its phenol continuously. In rain water that had flowed for a short time over a tarred road, the authors found a phenol content of 10 p.p.m.; likewise in rain water and snow from the vicinity of coke plants. Black digesting or digested organic sludge from river bottoms has a tarry odor. If it is stirred up in water it gives off the same taste-producing substance. The authors have found that chlorine can be used without the slightest danger of taste in water from wells in river bottoms (infiltration galleries) if the natural biological conditions in the river and underground gravel are in proper order, even though the river water contains phenol. Biological conditions in the gravel are disturbed by frost and high water. Phenol pollution cannot be entirely controlled and it is therefore desirable to employ processes for removal of the chlorophenol taste from the drinking water itself. SAUER describes the use of charcoal for disinfection and purification of water in German patent 400,128. WOLFF used activated charcoal filter to sterilize water in the tropics. ADLER proposed heavy over-chlorination (3 p.p.m.), allowing reaction period of 30 minutes, by which the organic matter is oxidized, and removing the excess chlorine with an activated charcoal filter. The authors have verified ADLER's

investigations and have found that loading of the filter with organic matter and other impurities like iron and manganese must be avoided. The charcoal filter may only be used for completely purified water. In the first experiment a strong chlorophenol taste was imparted to water by addition of 1 p.p.m. of chlorine and 1 p.p.m. phenol. Activated charcoal of 2 mm. grain size in a glass filter took up 1.5 to 2.0 percent of its weight of phenol before taste appeared in effluent. The filter can be regularly returned to its original efficiency by steaming out the phenol for 15 minutes. In an experiment on a larger scale, with a filter of 20 liters capacity, the first taste appeared after 2500 cubic meters of water had passed through, equivalent to 125 cubic meters of water purified per liter of charcoal. The rate of filtration was 6 cm. per second. Following this the Ruhrverband arranged a plant scale experiment at one of its city water works having a daily capacity of 25,000 cubic meters, situated below a charcoal factory, which had experienced taste troubles for years. The works were equipped with an artificial sand filter. It was necessary to place the charcoal filter in the pressure line following the pumps under an operating pressure of 15 atmospheres. The cross-section of the filter is 3 meters, the depth of filter material 2.5 meters, and the filter content 20 cubic meters. The charcoal was placed between 2 perforated plates having openings 21 mm. in diameter. A fine copper sieve was placed upon the lower plate and a 5-cm. charcoal layer composed of 4-mm. grains placed upon it; then a second layer 5 cm. deep composed of 3-mm. grains. In the upper portion the grain size is 2 mm. The charcoal is held in place by the pressure of the upper perforated plate, the water flowing upward through the filter. Provision was made for releasing the air trapped beneath the filter. Steaming, which to date has not been necessary, is provided for by a steam line connection to the dome of the filter, the steam passing downward through the charcoal. The cost for a daily capacity of 25,000 cubic meters [somewhat over 6.5 million U. S. gallons] amounted to about 20,000 marks, of which half was for charcoal. [With the mark at 23.8 cents, the cost would be at the rate of approximately $720 per million U. S. gallons capacity—ABSTRACTOR.] Good results have been obtained during the several months the filter has been in operation. The raw water has had its customary earthy, musty odor, but the filtered water tastes as fresh as spring water. Apparently, in addition to chlorophenols, all other unpleasant taste-producing substances, such as those from algae and mineral constituents, are also removed by activated charcoal.—*R. E. Thompson.*

Hamilton, Scotland, Water Supply. W. H. PURDIE. J. Instn. Munic. Co. Eng., Lond., 1929, **54**: 250. Surface supplies from neighbouring moorland and agricultural districts are collected in five reservoirs, Kype moorland supply being largest, purest, and most important. The Wellbrae and Cadzow Supplies are obtained from agricultural zone and are therefore more polluted. Total daily consumption of about 2,350,000 gallons is adequately provided for by total reservoir capacity of over 315 million gallons. Kype supply is colored, but free from suspended matter and with low bacterial content of 48 colonies per cc. Blackbog filters, consisting of four slow sand units, operated at rate of 3 inches per hour (about 2 million U. S. gallons per acre per day),

are used to purify this supply. Color is reduced from 16 to 14 (platinum standard), and bacterial content to 19 colonies per cubic centimeter. Water in Wellbrae and Cadzow Reservoirs, which is highly colored and somewhat polluted, is treated at Townhill, where original slow sand filters have recently been replaced by three rapid sand units of Jewell type, with coagulation and sedimentation basins, the only filters of this type in Scotland. Weston rate-controllers have proved very efficient. Washing is at rate of 7 gallons per square foot (about 14 inches) per minute, sand being thoroughly agitated by rakes meanwhile. Five percent solution of filter alum is added to raw water with automatic coagulant feed apparatus. Filtration reduces color from 9.5 to 1.2 (platinum standard), and bacteria from 254 to 4 per cubic centimeter; *B. coli* are absent in 50 cc. of filtered water. Diagrams and photographs of Townhill filters are appended.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Water Problems in France. Engineer, 1929, **147**: 700. Congress on Inland Waterways organised by Ligue Générale pour l'Aménagement et l'Utilisation des Eaux. SENTENAC described Paris water supply. Some water is taken direct from Seine and Marne and used unfiltered for municipal and other services. With this exception, supply is from spring water supplemented by filtered and sterilised water from Seine and Marne. Spring water supply is insufficient and river supply limited by needs of navigation; so that plans have been made for drawing water from underneath old bed of Loire. Large barrage reservoir will be constructed to control Loire. Commission has been formed to consider effect on Loire of this proposal. SUGUET dealt with barrage reservoirs to be erected in upper valley of Seine. Visits were paid to St. Maur and Ivry pumping and filter stations. The method of cleaning filter beds at St. Maur by water under pressure is described. St. Maur uses mainly chlorine for sterilisation, but uses also an elaborate ozone plant which, owing to its cost, is regarded as experimental. The varied equipment of the pumping station is described. Reference is also made to new water tower at Montmartre.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

The Hydrology of the Portsmouth District. D. H. THOMSON. Engineering, 1929, **128**: 55. Read at Annual Meeting of British Waterworks Association, July 2–5, 1929. From record, almost unbroken over period of 94 years, of well at Compton, much information regarding underground water conditions of the Downs can be gathered. Wells at Havant and Bedhampton which supply Portsmouth with water, are in a restricted area on which underground drainage from extensive tract converges.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

The Biological and Chemical Purification by Aëration. F. SIERP. Das Essener Heft. Kl. Mitt. der Landesanstalt fur Wasser-, Boden-, u. Lufthyg., Supplement **5**: **1927**: 133. Self-purification of sewage and, arising therefrom, the self-purifying process in rivers are discussed and work of many investigators on absorption of oxygen from the air is referred to. Natural biological

process of oxygen formation by plants and phytoplankton is described. Permissible stream pollution must be proportioned to oxygen content. Processes for purification of pickling and wool washing waters are discussed.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

The Solution of Oxygen. A. S. PARSONS and H. WILSON. Paper 13 T, Public Works, Roads, and Transport Congress, 1927. Experiments on efficiency of oxygen absorption by liquids from air are described and question discussed as to whether oxygen is obtained from broken atmospheric surface, or from air bubbles. Laws of solution and work of ADENEY are quoted. Experiments were carried out in two 18-inch pipes, one 7 feet deep and the other, 25 feet, each supplied with a diffuser. As result of sampling difficulties, which are discussed, opinion was formed that water projected under pressure into air and against some solid object very rapidly absorbs oxygen. As result of further experiments, authors also conclude that theory that oxygen is not readily dissolved from air bubbles cannot be maintained.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

Treating Sewage &c. ELKINGTON, H. D. E.P. 312,754. Ill. Off. J. Patents. 1929, No. 2114, 3757. Sewage or industrial waste waters may be purified by rendering alkaline (to pH between 8 and 11), precipitating colloids by adding electrolyte, and precipitating matter in suspension by adding coagulant. Lime may be used as alkaline reagent, calcium sulphate as electrolyte, and ferrous sulphate as coagulant. The three materials may be added to sewage as mixture. Sludge obtained from waste acid liquors from pickling vats of steel mills and consisting mainly of calcium carbonate, calcium sulphate, and ferrous hydroxide, can be used as major portion of treating mixture, with lime and ferrous sulphate added to secure requisite amount of alkali and coagulant. Dust from electrical or mechanical precipitators employed in manufacture of cement can also be used for same purpose, and dried and pulverised matter from settling tanks used for clarifying various waste liquors from alkali plants for producing chloride of lime, chlorine, and caustic soda is also a suitable material. Calcium chloride, barium chloride, strontium chloride, osmium chloride, thallium chloride, thallium sulphate, titanium chloride, thorium chloride, and zirconium chloride and sulphate are also mentioned as suitable electrolytes. Colloidal content of liquid under treatment may be preliminarily determined by dialysis carried out at low temperature with exclusion of oxygen.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

Purification of Waste Waters from Beet Sugar Factories. H. M. WILSON and J. H. GARNER. Surveyor, 1928, **74**: 185; Wass. u. Abwass., 1929, **26**: 3, 87. Insufficient purification of many beet sugar factories' effluents and cost of good clarification. Authors urge closer study of modern technique.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

Treatment of Waste Waters from the Beet Sugar Industry. J. T. TRAVERS. A.P. 1,711,105. Chem. Zbl., 1929, 2: 4, 502. Precipitation of organic matter by means of alkali (e.g., lime) and electrolyte (such as calcium sulphate, magnesium sulphate, or iron sulphate). Of special value is addition of cement dust. To 1000 parts of waste water there should be added 7 to 8 parts of a mixture of 40.8 percent cement dust, 17.1 percent $CaSO_4$, 30 percent calcium hydroxide; 12.1 percent $FeSO_4$. This will precipitate proteins and starches and coagulation will follow by formation of iron hydroxide.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Effect of the Effluents of a Sugar Factory on the Stream. MATSCHKY. Cbl. f. d. Zuckerindust., 1929, 37: 19, 520. Conditions at Ottmachau sugar factory are briefly summarised. Oxygen content was taken as measure of possible harmfulness to fish. Effluents, after settling, flow into the Neisze. Every two or three days during campaign, duplicate samples were taken from river at following stations: (1) just above effluent discharge; (2) 100 meters below discharge, and (3) 1 km. below discharge. One sample in each case was tested at once by WINKLER'S method and the other, after 24 hours. Results of tests are given diagrammatically and, for comparison, curve of Winkler's figures for oxygen saturation equilibrium in pure water at different temperatures and normal pressure. In 1927, river flow gave 36-fold dilution of effluent, but in 1928, only 28-fold. Increase of oxygen content at station (3) may be explained by bend in river between (2) and (3), where shallows act like weirs in promoting aëration. Tests show that river is not unduly polluted by effluents from sugar factory.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Solution of Phenol Troubles in the West Virginia-Pennsylvania Area along the Monongehela River. E. S. TISDALE. Proc. Lake Michigan Sanitation Congress 1928, 4: 3, 14. While provisions of Inter-State Stream Conservation Agreement between Ohio, West Virginia, and Pennsylvania were under discussion, West Virginia State University, State Health Department, and Fairmont Domestic Coke Corporation developed treatment process for phenol wastes. Plant erected at Fairmont works maintains efficiency of 97 percent phenol recovery and pollution of downstream water supplies has practically disappeared. Process is not expected to be financially burdensome.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

The Treatment of Spent Gas Liquors. A. C. MONKHOUSE. Conference of the Assoc. of Managers of Sewage Disposal Works, July 5 and 6, 1929. Conversion of ammonia liquor into salable products involves spent gas liquor effluent. Manufacture of ammonium sulphate is described and composition of effluents, spent liquor from still and "devil" liquor from gas coolers, is given. Noxious constituents are phenols, higher tar acids, thiocyanate, and thiosulphate. Percolating filter beds, matured by town sewage and then developed by addition to town sewage of gradually increasing quantities of

spent gas liquor, were found by experiment to deal satisfactorily with town sewage containing 8 percent of spent gas liquor at rate of 60 gallons per cubic yard per day. With two filter beds in series, 85 percent purification was obtained. Filter beds thus acclimatized can treat admixtures up to 8 percent of spent gas liquor with purified effluent without loss of efficiency. Preparation and use of filter beds installed at Foleshill Gas Works, Coventry, England, are described. Logical procedure on grounds of space and cost is, however, to treat liquor with sewage at sewage works. Evaporation of "devil" liquor reduces work of purification by about 20 percent. Phenol removal by volatilisation, by washing with solvents, and by activated carbon is referred to. Chemical oxidation is too expensive. Effluents Committee of Institution of Gas Engineers has paid particular attention to possible modifications in processes of gas manufacture with view to reducing proportion of noxious constituents. Separation of tar by precipitation from hot gas before condensation lowers amount of higher tar acids and, therefore, of color in liquor; but monohydric phenol content and total oxygen absorption value are increased. Methods for reducing thiocyanate and thiosulphate content have been introduced with success. Production of ammonium sulphate is no longer very profitable and disposal of effluent liquors constitutes a further burden on process.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

The Effect of Industrial Wastes on Filtration. J. R. BAYLIS. Proc. Lake Michigan Sanitation Congress, 1928, **4**: 3, 4. List is given of industries whose effluents have been known to pollute water supplies, and those affecting Lake Michigan in vicinity of Chicago are specially noted. *B. coli* content of 1 per 1000 cc., annual average, and turbidity of 0.2 p.p.m. are necessary standards for drinking water. Efficiencies of coagulation, filtration, chlorination, and softening by excess lime treatment are discussed. Where phenol pollution is extreme, even excess lime treatment without use of chlorine will not prevent tastes. Problem of liability for cost of eliminating pollution is discussed.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

The Disposal of Industrial Wastes. E. S. CHASE. Proc. Lake Michigan Sanitation Congress, 1928, **4**: 3, 10. Brief discussion of composition of industrial wastes, their effect on streams, sewers, and sewage treatment, special methods of treatment, and question of legal control of pollution.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Disposal of Effluents in the Textile Industry. C. H. MÖLLERING. Z. angew. Chem., 1929, **42**: 424. Effluents from cloth manufacturing, cloth-finishing, and wool washing, which are notably high in soap and fat, when discharged directly into streams, do not lead to pollution, but recovery of soaps and fats is economically desirable. Suitable processes for this purpose are known, but are little used. Treatment of wool washing wastes, as typical textile effluents, is described. Raw wool is washed twice, first with clean water and secondly

with soap and soda solution containing organic fat solvent. First washings are evaporated and burnt residue, containing from 65 to 70 percent potassium carbonate, is sold as crude potash. Second washings contain about 0.2 to 0.8 percent wool fat, 0.2 to 0.12 percent soap, 0.01 to 0.04 percent soda, and organic and inorganic impurities including nitrogenous wool decomposition products resulting from alkali attack ranging from emulsions to true solutions, forming a colloidal system. Degree of emulsification of fat depends on temperature and on peptizing effect of soap present. The pH lies between 8 and 9, indicating that albumen present is in form of sodium proteinates. Both mechanical and chemical treatments are available. Most important chemical method is coagulation with sulphuric acid, which removes insoluble soaps and fatty acids, keratin decomposition products, emulsified fats, and certain colloids, some of which are adsorbed by soap decomposition products. Magnesium chloride is now being recommended as a coagulant, avoiding expense of neutralizing acid effluent. Fats extracted from dried sludge. Raw fats are further treated to obtain pure wool-fat, fatty acid, neutral wool fat, and *Adeps Lanae*. There are two mechanical purification methods. Foam process consists in formation of foam by blowing in air. Keratin decomposition products form film round bubbles, adsorbing emulsified soap-fat particles, and concentrating from 60 to 70 percent of total fats in foam. Disadvantage of this process is difficulty of separating fat from other impurities in foam. Various ways have been patented. One method is to treat foam with alkali, and heat in autoclave. Alternatively impure liquid may be decanted from foam and fat removed by coagulation. Second mechanical method is by centrifuging in powerful separators. G. de Laval plate centrifuge and Sharples super-centrifuge are the most efficient machines. First, with 6000 r.p.m., has greater capacity than the more sensitive supercentrifuge with 15,000 r.p.m. Fatty layer is first separated from effluent and then centrifuged and pure fat freed from water. This process also removes only about 60 to 70 percent of emulsified fat. The very finely dispersed phyto, or cholesterin, portion of the fat emulsion cannot be removed by any known method.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

Effect of the Effluents of a Sugar Factory on the Stream. MATSCHKY. Cbl. f. d. Zuckerindust., 1929, **37**: 19, 520.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

The Poisonous Property of Salts of Some Heavy Metals. G. EBELING. Z. f. Fischerei, 1928, **26**: 1, 49; Wass. u. Abwass., 1929, **26**: 3, 95. Experiments were made on poisonous effect of salts of heavy metals on rainbow trout. Copper salts were found to be most harmful, 0.5 p.p.m. being fatal to adult trout. Iron, as ferrous or ferric oxide, was dangerous at concentrations from 10 p.p.m. and zinc, at from 2 to 3.5 p.p.m. One part per million of aluminium sulphate did no harm, but 5 p.p.m. killed fish in 5 minutes. Precipitation of these salts also causes damage by coating banks and bed of stream so that normal plant development is hindered and fish disappear.—*M. H. Coblentz* (*Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board*).

Coal Mine Drainage for Purifying Streams. Colliery Guardian, 1929, 139: 526. Drainage from majority of coal mines is acid and deleterious to streams into which it flows. The Pittsburgh Experiment Station of the U. S. Bureau of Mines has studied the drainage from a considerable number of mines in the thick Freeport Bed in vicinity of Pittsburgh. These waters are strongly alkaline and aid in neutralising any harmful acidity. Alkalinity is due to underground water entering the workings containing more than enough alkali to neutralise all acid met with in mines. Acid water is sometimes present in one section of mine, but combined stream of effluent water is always alkaline.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

Stream Pollution and Industrial Waste Investigations of the United States Public Health Service. H. R. CROHURST. Proc. Lake Michigan Sanitation Congress, 1928, 4: 3, 25. Account of formation and activities of United States Public Health Service, which is carrying out studies of biochemistry of sewage and industrial wastes, pollution and natural purification of several watercourses, operation and efficiency of sewage plants and water purification plants, and types and treatment of industrial wastes.—*M. H. Coblentz (Courtesy of the Department of Scientific and Industrial Research, Water Pollution Research Board).*

The Function of the City Sanitary Engineer. JOEL I. CONNOLLY. Municipal Sanitation, 1: 1, 22, January, 1930. This comprehensive article includes several sections of interest and value to water sanitarians. *New facts on water supplies.* Hospital supplies, both of drinking and of sterilized water, have been found dangerously polluted by drainage or by siphonage from fixtures and sterilizers directly connected to distribution system, when pressure in latter becomes low. Stock supplies of sterile water may, in turn, become contaminated through leaky valves on sterilizers. Low water pressure may cause pollution by siphonage from trap seals and urinals if these are not constructed to prevent it. *Sources of typhoid affecting traveling public.* An undetermined, but relatively small, proportion of vacation typhoid is probably contracted aboard trains, vessels, and vehicles obtaining water from city supplies. Preventive measures include water treatment aboard carrier; co-operative control by municipality and by U. S. Public Health Service; special water connections at docks, piers, and levees; restriction of use of toilets and of navigation in zones of municipal water intakes; and inspections of water and sewage connections of coaches in railroad terminals. *Water purification.* The engineer should assure himself periodically that crib (water-intake) tenders are not intestinal disease carriers and that suitable disposition is made of their sewage. He should thoroughly familiarize himself with purification system especially with its weak points, in order to cope successfully with emergencies. Assurance should be had that plant and distribution samples are systematically collected and analyzed by standard methods of the A. P. H. A. *Prompt report of tests essential.* Presumptive as well as completed *B. coli* tests should be promptly reported to the engineer. *Controlling chlorination.* Residual chlorine by *o*-tolidin test should be determined from

time to time at points in distribution system and hourly, or oftener, at dosage point. Constant vigilance is necessary in order to maintain residual chlorine above prescribed minimum; any changes should be immediately reported to engineer in charge. *Construction and control of swimming pools and pollution of water supply by sewage and trade wastes* are matters which properly belong under jurisdiction of city engineer who should co-operate with state sanitary engineer.—*R. E. Noble.*

Swimming Pools Abroad are Well Protected. Municipal Sanitation, 1: 1, 36, January, 1930. Contents of 370,000-gallon swimming pool at Hammersmith, London, England, are completely circulated every 6 hours. Water is treated with alumino-ferric and ammonia, filtered, aërated, and chlorinated (0.5 p.p.m.).—*R. E. Noble.*

Ohio Stream Pollution to be Studied. Municipal Sanitation, 1: 1, 96, February, 1930. F. H. Waring, chief engineer of Department of Health, and Prof. John C. Prior, of Ohio State University, have been assigned to investigate problem of stream-pollution control. Objects are: (1) to present existing status of stream pollution; (2) to report upon corrective measures now in effect; (3) to devise reasonable measures for further amelioration. Stream-pollution will be investigated from three different standpoints: (1) effects upon public water supplies derived from surface sources; (2) abatement of nuisances to public health or comfort; (3) conservation of Ohio streams to effect greater public benefits. Further plans include consideration of pollution agencies in order of their importance, such as municipal sewage and industrial wastes, and of their natural or semi-natural effects.—*R. E. Noble.*

Higher Court Decisions. Injunction Against Stream Pollution. Municipal Sanitation, 1: 10, 576, October, 1930. Meriwether Gravel Co. *v.* State, 26 S. W. (2d) 57. Defendant discharged water used to wash gravel into nearby stream, sullying same. Injunction granted.—*R. E. Noble.*

Notes on the Determination of Biological Oxygen Demand. (Abstracts of paper presented before Central States Sewage Works Assn.) William D. Hatfield. Municipal Sanitation, 1: 10, 581, October, 1930. Standard methods (A. P. H. A.) procedure for this test is tedious and calls for elaborate equipment. This results in great lack of uniformity throughout the country and is conducive to error. Dilution method and rubber stoppers are found unsatisfactory. Dilution method is, however, accurate to within 5 percent, provided that (1) incubation bottles are carefully cleansed with cleaning solution, rinsed thoroughly, and dried and (2) dilution water is not toxic to normal bacterial flora and is well stabilized in regard to its own organic content. Distilled water should not be used, because it is toxic and gives low results. Sodium bicarbonate solution in distilled water, 500 p.p.m., is much better, but gives lower results than natural waters. A standard synthetic dilution water should be adopted, to contain calcium, magnesium, sodium, bicarbonate, sulphate, and chloride ions. At Decatur, Illinois, the softened and chlorinated city tap water as dilution water gave very bad results.—*R. E. Noble.*

Hydro-Electric Engineering in Great Britain in 1930. Engineer, 151: 13, 1931. The important Lochaber plant for aluminium production, which will have eventual capacity of 120,000 h.p., entered into operation during year. Tunnel 15 miles long and of 15 feet effective diameter has been bored in solid rock and waters of 11 streams have been diverted by dams and led into tunnel. The two pipe lines are of lap-welded steel, of spigot and faucet type; all joints were electrically welded inside and out and not a single weep showed. Work has begun on Grampian works, designed for distribution of electricity, and part of a much larger scheme, involving construction of concrete dam 1360 feet long and 57 feet high, piercing of two tunnels 14,470 feet long, laying of 2 riveted steel pressure pipe lines 2700 feet long and 8 feet in diameter, and building of power house with initial capacity of 32,000 k.w. in 2 units.—*W. G. Carey.*

Water Power Resources of Africa. Engineer, 151: 47, 1931. Mr. L. B. Woodworth at South African Institute of Electrical Engineers states that while estimated potential water power of Northern America is 62 million horse power, that of Africa is 190 million. Of this amount, 44 million horse power is available in British possessions, but major portion is controlled by Belgium.—*W. G. Carey.*

Bursting of Water Mains. Metropolitan Water Board Report. 120 pp. Engineer, 151: 79–80, 1931. Metropolitan Water Board Sub-Committee report that causes of failure of mains are: (*a*) excessive vibration caused by traffic, when main is too near surface, (*b*) direct blow from traffic, transmitted through walls of a pit, or through solid concrete, placed immediately over main, (*c*) subsidence of ground, owing to deep foundations, or to pumping operations for reduction of level sub-soil water, (*d*) excessive depth of cover over unprotected main, (*e*) faulty castings. Steel pipes adequately coated to resist corrosion are recommended for trunk and for large high-pressure mains.—*W. G. Carey.*

A Water Plant to Include Carbonation, Softening and Filtration. L. R. Howson. Water Works Eng., 84: 3, 157, February 11, 1931. Cedar Rapids, Iowa, new water works takes its supply from Cedar River. Low-lift station is located 4,500 feet upstream from main purification plant and is operated by remote control. Aëration is through nozzles of "Sprayco" type, each having discharge capacity of 115 g.p.m. at 15 feet head. Chemicals are received in bulk and are unloaded by vacuum system. Four dry-feed machines are provided. Chemicals are added in premixing basin, having storage period of two minutes, agitation being effected by baffling supplemented by air-discharge through perforated pipes at bottom of basin. From pre-mixers, water passes to two mechanically operated stirring basins. Settling basins have detention of two hours, at rated plant capacity, and are arranged to be operated in series, or parallel; normally, however, in series on account of practicability of returning for use in second basin sludge recovered from the first. Water is recarbonated with carbon dioxide generated by oil burner under boiler. Four mechanical gravity filters, of 3-m.g.d. capacity each, treat re-

carbonated water. Depth of sand is 30 inches and of gravel, 21 inches. Underdrain system consists of manifold with laterals perforated with ⅜-inch holes at 6-inch centers. Clear well of 600,000 gallons capacity is directly under filters. Building is of Gothic type with brickwork selected to match the stone trim.—*Lewis V. Carpenter.*

When Water Rent Becomes a Tax. LEO T. PARKER. Water Works Eng., 84: 3, 161, February 11, 1931. Bills for water furnished by municipal water companies become collectable as taxes only after special legislative enactment to this effect. Courts usually hold that water company cannot shut off water when payment of a bill is held up during litigation. When municipality purchases land for city purposes, it becomes tax free and court cannot collect amount of past due taxes from previous owner. Street railway is not liable for payment for street improvements between its tracks, unless it specifically agrees to the improvement. This decision has been made many times, in spite of state laws and municipal franchises to the contrary. It has been held that when bathing in a pond, or stream, from which a city takes its water supply, may contaminate the water, such bathing, even by lessee of cottage and land abutting on pond, is not reasonable use of riparian right. Higher court held manufacturer of water faucet liable in damages when handle broke and seriously cut hand of person turning on water. Landlord was held liable when water froze in an upper story and destroyed merchandise on first floor, because landlord did not provide tenant with means of cutting off water.—*Lewis V. Carpenter.*

Legal Aspects of Water Pollution. LEO T. PARKER. Municipal Sanitation, 2: 3, 129–131, March, 1931. *What is Police Power?* In its broadest significance, it is sovereignty itself. In its narrower sense, it signifies the governmental power of conserving and safeguarding public safety, health, and welfare. Its scope is not unlimited, and validity of any mandate promulgated under it is for determination of the courts. *Surface Water Pollution.* It is well settled that owner of upper land cannot be prevented from permitting natural flow of water from his land, although it may be such as to pollute lower land, or streams. This rule applies to natural flow only, as from rain, snow, springs, and drainage without any defined channel. *Governmental and Ministerial Functions* vs. *Water Pollution.* A municipality will not be held liable in damages for negligence in the exercise of its governmental powers of a public nature; but it is liable for damages caused by neglect of its private duties, which are ministerial. *City Liable for Water Pollution.* Only where business, or work, of the municipality is confined to exercise of a strictly governmental function, free from any active wrong-doing on its part, is immunity from responding in damages to persons injured through negligence of its public officers, agents, or servants, warranted. *When Cemetery is Nuisance.* If cemetery undoubtedly will pollute municipal water supply, it is a legal nuisance and Court will enjoin interring bodies therein. In order for such injunction, it must be proved conclusively that germs therefrom will pollute water. *Damage Allowed for Water Pollution.* A railway train was wrecked. Several tank cars of oil burst and the ground became saturated with

oil. Railway company failed to dig into the ground and eliminate spilled oil, result being that it seeped for several months into a nearby stream, polluting same. Cases are cited under all but one of above headings.—*R. E. Noble.*

Improved Drinking Facilities in Places of Employment. MARIE CORRELL. Municipal Sanitation, 2: 2, 71, February, 1931. Types of drinking facilities found in many places of employment and lack of regulation of this problem by most States indicate that more effective action is needed to prohibit use of common cup and to prevent installation of equally unsanitary fixture in its place. Although for over 13 years it has been known that vertical-jet fountain is unsanitary and that better designs are available, yet vertical-jet fountains are still manufactured and installed. Health officials and sanitary engineers have made efforts to correct this condition, by sponsoring adoption of three standards for sanitary drinking fountains; but since in only 18 States and in District of Columbia is use of angle-jet rather than vertical-jet fountains officially recommended, effort should be made to secure more general adoption of these standards in interest of public health.—*R. E. Noble.*

Endemic Goitre and Myxœdema. J. WAGNER-JAUREGG. Wien. Klin. Woch. 1930, 5: 43, 1–6. From Bulletin of Hygiene, 5: 12, 941, December, 1930. Etiology of endemic goitre remains obscure, but there is no doubt that it can be prevented by supplying regularly small quantities of iodine. Inverse relation between amount of iodine in soil and water of a district and prevalence therein of endemic goitre frequently holds good; but exceptions recorded make it difficult to regard iodine deficiency as only exciting cause. Theory of bacterial pollution of drinking water will sound convincing only if specific organisms are isolated which occur in endemic regions and are absent in regions free from goitre.—*Arthur P. Miller.*

Relationship of Iodine in Soil and Drinking Water to the Chronic Hypertrophic Type of Endemic Goitre. R. MCCARRISON. Eastern Assoc. Trop. Med., Trans. Seventh Congress, British India, 1927, 5: 3, 455–6. From Bulletin of Hygiene, 5: 12, 942, December, 1930. In Himalayan India, endemic goitre is more common in iodine-poor than in iodine-rich localities. However, it is possible for goitre to be widespread in villages where water supply contains relatively large amounts of iodine, while, on the other hand, very minute quantities of iodine may be found in water of villages where goitre is extremely rare. Essential cause of disease is conjectured rather to be toxic action of bacteria present in intestinal tract than iodine deficiency; but latter favors its development.—*Arthur P. Miller.*

Goitre in School Children. The Incidence of Goitre in School Children in Relation to the Amount of Iodine in Soil and Water in Certain Districts of the North Island of New Zealand. R. A. SHORE and R. L. ANDREW. Departments of Health and Scientific and Industrial Research. 1929. Wellington. From Bulletin of Hygiene, 5: 12, 942, December, 1930. Careful examination was made of relation between incidence of goitre and iodine content of soil. Report of this work gives results of soil and water analyses in various districts

and incidence of goitre among children therein and discusses relation between observed incidence and iodine-content in soil and water. In most instances, high iodine content was concurrent with low goitre incidence and vice versa. Exceptions to this were noted. Analyses of soil from many parts of world indicated that usual iodine content varied from 6 to 60 parts in ten million. In New Zealand work, values of 10 to 200 parts in 10 million were considered low, and values above 400, high.—*Arthur P. Miller.*

Advisory Committee on Water. Second Report of Legislation Sub-committee. Ministry of Health. A. R. ATKEY, A. B. E. BLACKBURN, A. E. CORNEWALL-WALKER, F. W. MACAULEY, W. TERREY. 1929. London. H. M. S. O. From Bulletin of Hygiene, 5: 12, 969, December, 1930. Attention was given to definitions of "domestic supply" and "supply pipes." Recommendation was made to secure powers for penalizing waste of water and proposals were advanced to revise financial provisions of Act of 1847. Amendments to Waterworks Clauses Acts were also offered.—*Arthur P. Miller.*

St. Lythans Reservoir. Urban District of Barry, South Wales, Inaugurates New Water Supply Scheme. ANON. Water and Water Engineering, 32: 381, 461-463, October 20, 1930. In response to need for additional water in high pressure district of Barry, where yield from local wells is insufficient, Taf Fechan Water Supply Board arranged to make good the deficiency. Article describes the new works, including pipe lines and reservoir, necessary to bring the new supply to area needing it.—*Arthur P. Miller.*

Kniveden, England, Reservoir. Proposed Construction under the Leek Urban District Council Water Act, 1925. W. E. BEACHAM. Water and Water Engineering, 32: 381, 464-467, October 20, 1930. Under previously procured authority, Leek Urban District will build additional, 1-m.g. concrete reservoir which will be used to supply higher part of district, consuming about 300,000 gallons per day. Design of reservoir, embodying usual principles involved in such work, and necessary pipe lines tying new reservoir into general water system, are described.—*Arthur P. Miller.*

Waterworks Construction. III. Pipe Laying. M. RATCLIFFE BARNETT. Water and Water Engineering, 32: 381, 470-472, October 20, 1930. In forecasting delivery of water by pipe line at intervals during a period of years after laying, it is unwise to formulate generalizations based upon a few special cases. Although small diameter pipes may lose much carrying capacity due to incrustation, large pipes (40-inch to 44-inch) do not suffer in same proportion, because incrustation does not continue to grow, but remains stationary after reaching thickness of from 1 to 1½ inches. Author contrasts theoretical deductions as to carrying capacity with instances under his constant observation. In one case, four year old pipe was found to deliver 50 percent in excess of calculated capacity, instead of theoretical 21.7 percent less. Reduction in carrying capacity may be caused by constructional, or other unknown obstructions in pipe line. Valves of insufficient diameter exert serious retarding influence, particularly when reduction in diameter is abrupt.—*Arthur P. Miller.*

Sand Spun Cast-Iron Pipes. A New Development by the Staveley, England, Corporation. Anon. Water and Water Engineering, 32: 381, 472–474, October 20, 1930. Article describes manufacture of cast-iron pipes in sand-lined molds by centrifugal process. Advantages are: (1) impossibility of pipes being chilled, making annealing unnecessary; (2) elimination of cores; (3) production of homogenous and close-grained metal; (4) elimination of strain, by leaving pipes in molds until quite cool; and (5) additional strength with less metal, thereby reducing weight and permitting longer lengths.—*Arthur P. Miller.*

Chemical Precipitation of Humic Pigments. Dr. C. P. Mom. From Water and Water Engineering, 32: 381, 476–477, October, 1930, and 32: 383, 526, November 20, 1930. In swampy coastal regions of East Sumatra, only very darkly colored peat water is available in any quantity. Bagan Si Api Api, an important town in this area, with fire system using this water, desired also to have piped supply of suitable drinking water. This led to experiments on chemical purification of the water. Results may be summarized as follows. (1) Coagulation of humic matter by means of alum is effected by simultaneous influence of hydrogen ions, aluminum ions, and colloidal hydrous aluminum oxide. (2) With ferric salt, coagulation is accomplished by simultaneous action of hydrogen ions, ferric ions, and colloidal hydrous ferric oxide. (3) If free chlorine is present, part of organic matter fails to coagulate because of destructive effect of chlorine on humus colloids.—*Arthur P. Miller.*

Chelmsford's, England, Water Supply. Opening of New River Works. Anon. Water and Water Engineering, 32: 381, 411–415, September 20, 1930. Instead of drilling additional wells, Chelmsford's new supply is being taken from River Chelmer. Purification, based primarily on experience of Southend Waterworks Company with similar water, is by treatment with alum and excess lime, followed by recarbonation and pressure filtration.—*Arthur P. Miller.*

Removal of Lead, Zinc, Copper, and Tin from Drinking Water by Means of Base-Exchange Materials. B. A. Adams. Water and Water Engineering, 32: 381, 415–419, September 20, 1930. Experimental work, performed under auspices of Water Pollution Research Board (England), shows that traces of lead, zinc, copper, and tin salts in solution are exchangeable with base-exchange materials and that presence of excessive amounts of calcium, magnesium, sodium, and other ions usually present in drinking waters do not affect the process. Except with copper salts in presence of citric acid and citrates, acids and alkalis do not interfere. Such copper salts may be removed with manganese permutite. The amount of either lead, zinc, or copper, exchangeable with "Kenzelite," a glauconite material, equals its calcium exchange value. Regeneration can be effected with ten percent solution of sodium chloride in case of all salts mentioned but those of lead, for which sodium nitrate is needed.—*Arthur P. Miller.*

Harpenden, England, Water Tower. An Improved Method of Construction. Anon. Water and Water Engineering, 32: 381, 419–421, September 20, 1930.

Reinforced concrete, as compared to steelwork, has disadvantages both of higher initial cost and of being slower to erect. Assemblage of loose bars and stirrups in reinforced concrete is further source of anxiety. These disadvantages are overcome in "reinforced steelwork," as exemplified in new Harpenden water tower. The steelwork in this design forms rigid, self-supporting, pre-erected structure, built up of R. S. sections and plain round steel rods, which will carry its own weight, that of the attached, or suspended, concrete forms, and that of the wet concrete and other loads incidental to construction. Unlike structural steel construction with concrete protecting layer, the rigid steelwork in this scheme will not carry the ultimate load until it has been reinforced with concrete. This system is suitable for water-containing structures, because full permissible tensile stress in steel can be attained without detriment and positions of stoppage planes so fixed as not to affect water-tightness.—*Arthur P. Miller.*

Damage to Water Pipes Caused by Frost. Royal Institute of British Architects Sub-Committee's Inquiry and Recommendations. Anon. Water and Water Engineering, 32: 381, 423–424, September 20, 1930. As a result of severe frost in England (12°F. in London) in 1929 and consequent damage and inconvenience from frozen fixtures, it is recommended that local water companies and authorities should make by-laws putting water pipe lay-outs on a more scientific basis, and certain constructional data and suggestions relative to plumbing are submitted for use in this connection.—*Arthur P. Miller.*

The Use of Active Carbon for the Purification of Waters. Recent Developments Reviewed. Anon. Water and Water Engineering, 32: 381, 424–426, September 20, 1930. At Hamm (Germany) active carbon filter plant was installed last year. Carbon used was Hydraffin II. Carbon bed 2½ m. thick is supported in 3 m. container on sieve. Water flows through filter at rate of 3½ cm. per second (84 inches per minute); but later experiments at other places indicate a better rate to be from 1 to 2 cm. per second (24 to 48 inches per minute). Flushing filter backwards once a month brings it back to original speed. Purifying action is apparently not due to pure adsorption, but to decomposition of chlorophenol and removal of harmless decomposition products. After seven months filter was rejuvenated by steaming and thereafter, at shorter intervals, until after 1¼ years carbon was replaced with new. At Dresden, carbon filter is used to remove unpleasant taste and at Magdeburg, small filters, filled with fine-grained carbon. At Aussig, carbon is used as dechlorinator after superchlorination. Stuttgart has decided to follow Aussig practice which will also effect color improvement in finished water. Carbon is to be regenerated with strongly chlorinated soda and calcium chloride solutions.—*Arthur P. Miller.*

NEW BOOKS

Untersuchung Des Wassers an Ort und Stelle (The Investigation of Water at the Source). H. KLUT, sixth edition, Julius Springer, Berlin, 1931. This handy little volume of 180 pages has again been revised and brought up to

date. It has also been considerably enlarged since the appearance of the fifth edition in July, 1927. The book contains detailed instructions for the collection and examination of water samples. A very extensive list of references to the original literature is given in connection with each section. The present edition contains a valuable 40-page summary of our knowledge of corrosion. This chapter is a welcome addition to the present edition. The generous list of illustrations of apparatus adds materially to the value of the work. The American chemist having to deal with water analysis will find this a very valuable handbook, supplementing at many points American publications in this field.—*A. M. Buswell.*

The Ammonia-Chlorine Process and Its Application in the Treatment of Water. JOHN C. BAKER, FRANZ C. SCHMELKES in collaboration with Wallace & Tiernan Research Staff. Research Publication 305: Wallace & Tiernan Company, Inc. The hypochlorous acid formed when chlorine dissolves in water,

$$Cl_2 + H_2O = HCl + HOCl \quad (1)$$

reacts with ammonia to form, dependent upon pH, either monochloramine (NH_2Cl), dichloramine ($NHCl_2$), nitrogen trichloride (NCl_3), or mixtures of these, the reactions being:

$$HOCl + NH_3 = NH_2Cl + H_2O \quad (2)$$

$$2HOCl + NH_3 = NHCl_2 + 2H_2O \quad (3)$$

$$3HOCl + NH_3 = NCl_3 + 3H_2O \quad (4)$$

"The reaction (4) takes place at pH 4.4, or less, whereas above pH 4.4 a gradually shifting equilibrium occurs in which dichloramine and monochloramine coexist, 100 per cent dichloramine being produced at pH 4.5 and 100 per cent monochloramine at pH 8.5." Chief difference between action of chlorine and of the chloramines is probably of electro-chemical nature. Oxidation potential of chlorine is considerably higher than that of chloramines. Extended discussion of this phase of subject is given. Points of application of ammonia and chlorine should not be so far apart as to permit dissipation of ammonia by organic life, but should be sufficiently far apart to permit thorough mixing of ammonia with main body of water, before chlorine is added. Diagrams are presented showing graphically effects of application of the two chemicals at different relative distances. Ammonia-chlorine process is a preventative of tastes produced by chlorine. Chloramines do not produce taste because of their lower oxidation potential. "A certain oxidation potential is necessary to introduce chlorine into the phenol molecule to form the taste-producing chlorophenol compound. Any means taken to lower this oxidation potential will bring about taste reduction. Ammonia by combining with chlorine lowers the chlorine oxidation potential. Increase in pH likewise lowers the oxidation potential, but not sufficiently to prevent taste-producing reactions. It is no mere coincidence that the center of phenol taste epidemics in the country is in an area of water supplies with lower pH." Special attention is called to the slow action of chloramine as a sterilizing agent and its variation directly with the pH. Care should be taken to insure that adequate time is allowed for sterilization before water reaches consumer. Where prechlorination and preammoniation are practiced, sufficient time elapses during subsequent course

through plant, but where these processes can be used only on plant effluent, caution is particularly advised. High residual should be carried and from one-half to one hour interval should occur between final treatment and earliest delivery to consumer. Persistency of the residual has given remarkable results in reduction of after growth, algae, and crenothrix. This paper merits close study from any one interested in ammonia-chlorine process. It concludes with bibliography giving original sources of all references.—*R. L. McNamee.*

Selected Papers, University of Florida, Short Course in Water and Sewage Treatment. General Extension Division Record, 11: 16, April 1930, University of Florida, Gainesville, Florida. **Cement Lining of Salvaged Cast Iron Pipe.** J. R. TANNER. 12–16. Unlined cast iron pipe supply main to Palm Beach across Lake Worth, laid in 1920, was abandoned in 1927 due to many breaks and leaks resulting from tidal action. It was salvaged in 1929 and 1930, at total cost of about 90 cents per foot. Salvaged pipe was brushed out and lined with cement by centrifugal process similar to that used at factory. Cost of lining was 19 cents per foot. **The Hardness of Public Water Supplies in the United States.** W. D. COLLINS. 17–19. Brief résumé of subject matter of U. S. G. S. Water Supply Paper 496. **Feedwater: Its Chemical Treatment as Practised on the Atlantic Coast Line Railroad.** O. L. ANDERSON. 19–20. Typical water treatment plant on A. C. L. employs hydrated lime, soda ash, and sodium aluminate and consists of automatic proportioner, settling tank having detention period of four hours, and excelsior filter. Typical water available in Florida carries incrusting solids up to 50 grains per gallon. Experience in operation of these plants indicates that close attention and frequent general supervision are essential to good results. Every pound of scale in boiler feedwater is calculated to increase operating expenses 13 cents. A. C. L. is now experimenting with internal boiler treatment, especially for stationary steam generation. **The Daytona Beach Water Softening Plant.** B. F. TIPPINS and F. T. STUART. 21–23. **Removal of Iron from City Water Supply, Punta Gorda, Florida.** CHARLES K. DODD. 23–27. Punta Gorda supply, obtained from five 6-inch wells, is aërated and filtered for iron removal. **The Boca Raton, Florida, Water Treatment Plant.** H. S. R. MCCURDY. 28–31. Boca Raton is supplied by two 10-inch wells driven to rock. Treatment plant consists of aërator, mixing tank, coagulation basin, carbonation basin, and filters. All units except filters are designed for rate of 1 m.g.d.; present installation of filters for ½ m.g.d. **The Acquisition of a New Source of Raw Water Supply for Orlando.** C. F. UNDERHILL. 31–35. Interesting discussion of problems presented in acquisition of rights to use of Lake Underhill for new supply for Orlando. **Accounting for Municipal Water Works.** BEN BRIDGES. 35–38. Outline of accounting system developed at Tallahassee, Florida, embracing four public services, electric light, electric power, gas, and water. Ledger sheets and consumers' bills carry statements of all four services. Meter records and work orders described and illustrated. **Water Works Accounting in Privately Owned Plants.** P. PAUL DEMOYA. 39–43. Discussion of system in use at Stuart, Florida, with particular reference to operating and maintenance expense accounting.—*R. L. McNamee.*

Developments in the Treatment of Acid Mine Drainage. LEWIS V. CARPENTER and ALFRED H. DAVIDSON. Reprint Bulletin of the West Virginia University Scientific Association. Drainage from coal mines has become an acute problem in West Virginia due to acidity of water. From numerous measurements it is indicated that quantity of mine drainage amounts to about 25 per cent of rainfall on area from which coal has been exhausted. For an ordinary year in West Virginia it is about 1,000 gallons per day per acre of coal exhausted. There is little seasonal fluctuation. No direct relation is observed between quality of coal and acidity of drainage water. Acid mine water is very detrimental to streams used for domestic, commercial, or recreational purposes. It is supposed to have serious effect upon fish life, but further research is necessary to determine limiting concentrations. No one method of treatment has been found satisfactory for all mine waters. Largest installation [presumably, in West Virginia, ABSTR.] is at Calumet mine of H. C. Frick Company. Water is pumped from mine, lime mixed with it, and treated water passed through thickener having retention period of four hours, following which there is period of plain settling. Settled sludge is pumped on to revolving steam-heated drum which evaporates large part of water and leaves deposit of ferric hydrate. Plant has been discontinued because of inability to market ferric hydrate. Other plants of this type have been designed but not built. Filter employing limy marl as neutralizing agent, installed at one plant, improved character of stream and abundance of fish life very much, but has, nevertheless, been abandoned. Various schemes for neutralization of acid mine waters have been patented. One of these is based on passing liquid waste through porous calcium carbonate and maintaining contact period of between two and five minutes. Kaplan-Reger process for treating mine waste waters consists "in adding to the sulfate waters a complex organo-metallic compound which combines with the chemical compounds of the water and forms a blue pigment to be known as Monongahela blue and at the same time removes the acid . . . the added chemicals combine completely with the chemicals in the water to form an insoluble marketable product, leaving the water practically free from acid and mineral substances." No large scale installations have been built. Magnesite has been suggested as neutralizing agent. About one-third as much magnesite as lime is required. Abandoned mines yield large volume of acid water. This burden should be removed from streams by sealing such mines. In studies of the effect on pH value of dilution of mine water, it appears that "there is a range of dilutions for pactically every mine water which gives the same pH value. . . . Very frequently samples of water diluted by an alkaline stream showed the same pH value [after] as . . . before the addition of the diluting water." Research is necessary to determine the limits of pH and acidity essential for support of major forms of aquatic life. It is generally agreed that, after coal has been mined, the percolating water brings about the oxidation of pyrites into ferrous sulphate and free sulphuric acid. The ferrous salt oxidizes to ferric which, easily precipitated, gives the familiar brownish-yellow deposit of streams that carry sulphur water. The following reactions indicate what is assumed to take place:

$$2FeS_2 + 7O_2 + 2H_2O = 2FeSO_4 + 2H_2SO_4$$

and this reaction continues as follows:

$$4FeSO_4 + 2H_2SO_4 + O_2 = 2Fe_2(SO_4)_3 + 2H_2O$$

[and, by hydrolysis, $Fe_2(SO_4)_3 + 6H_2O = 2\ Fe(OH)_3 + 3H_2SO_4$]. Anaërobic activity of sulphur bacteria might possibly explain in some cases the high acidity found. "A series of laboratory tests were made to determine the feasibility of using various neutralizing agents for the treatment of acid mine waters. A sample of water with an acidity of 22,700 p.p.m. and a pH of 2.0 was selected for the tests. The neutralizing agent was added in varying amounts until the pH value had reached 5.6. This value is high when the volume of diluted water available is appreciable but it is about the lowest limit for the support of major fish and plant life. The quantity of sludge was measured, and its water content determined.

Data on the neutralization of 1000 gallons of acid mine water and the resultant sludge

NEUTRALIZING AGENT	POUNDS PER 1000 GALLONS	APPROXIMATE VOLUME OF SLUDGE	WEIGHT, DRY
		per cent	
Lime (90 per cent CaO)	172.5	35	424
Pulverized limestone	434.6	35	621
Sodium hydroxide	130.0	25	188
Calcium carbonate	161.5	40	352
Sodium carbonate	200.0	40	194
Ammonium hydroxide	160.0	20	130
Sodium carbonate	80.0		
plus lime	66.3	25	
Lime	61.9		
plus NaOH	60.0	35	183
Potassium ferrocyanide*	142.3		264
Lime	60.0		
plus potassium ferrocyanide	103.5		389
Sodium cyanide	140.0		188

* This quantity of potassium ferrocyanide brought the pH value to 3.0 only, as compared to 5.6 for the other processes.

On a cost comparison basis it was figured that the limestone treatment would be cheapest. On the basis of capitalized cost of plant drying of sludge and no return for the sale of sludge it was figured that the total cost per 1000 gallons would be $0.90. This treatment would neutralize the water but would not make it suitable either for domestic or industrial use on account of its hardness. It does seem to remove the majority of the iron from the water. These laboratory tests clearly indicate (as many other results have already shown) that the success of treating mine waters will not be assured until some method of utilizing by-products has been developed that will help pay the operation costs." Paper concludes with short bibliography.—*R. L. McNamee.*

Experimental Studies of Natural Purification in Polluted Waters. IV. The Influence of The Plankton on the Biochemical Oxidation of Organic Matter.

C. T. BUTTERFIELD, W. C. PURDY, and E. J. THERIAULT. Public Health Reports, **46**: 8, 393–426, February 20, 1931. Further comprehensive report from Division of Stream Pollution, U.S.P.H.S. Experiments were undertaken to determine interactions of the biological factors, bacteria and plankton, and part played by each in process of natural purification, especially of polluted streams. Preliminary work showed the necessity of determining, in a simple reproducible medium, extent of oxidation, if any, which occurs: (1) in absence of all living organisms; (2) in presence of pure, or mixed, cultures of bacteria in absence of plankton; and (3) in presence of pure cultures of plankton in absence of bacteria. With the authors' dilute dextrose-peptone medium, following results have been obtained: (1) When preserved free from biological activity, it does not absorb any dissolved oxygen under conditions of these tests. (2) When inoculated with bacteria in pure culture, it favors their growth. They multiply rapidly, reaching a limiting population by second day of incubation, and maintaining it for long periods of time. (3) While bacteria were actively multiplying, oxygen was depleted at rapid rate. After limiting population had been reached, depletion of dissolved oxygen practically ceased, although living bacterial population remained quite high. (4) Results observed with mixed cultures of bacteria, free from plankton, were same as with pure cultures, except that extent of oxidation was somewhat greater. (5) The protozoon, *Colpidium*, grew well in presence of bacteria in dilute dextrose-peptone solution; but not when bacteria were absent. When concentration of food in medium was increased 100- to 1000-fold, *Colpidium* grew well without bacteria. It is concluded that in dilute medium, bacteria act as collectors or concentrators of *Colpidium* food. (6) *Colpidium* growing in pure culture used up only comparatively small amount of oxygen. (7) When bacteria and plankton were grown together in dilute medium, results obtained during first two days of incubation were approximately same as when bacteria only were present. After first two days, bacterial numbers were not maintained, but were reduced rapidly; reduction being accompanied by plankton increase. Oxidation process did not cease, but continued as in natural polluted waters. (8) In general it can be said that extent of oxidation observed in dilute dextrose-peptone solution varied thus; that the greater the variety of organisms acting in the medium, the more extensive was the oxygen depletion observed. Theory is advanced that chief function of certain plankton in biochemical oxidation process is to keep bacterial population reduced below saturation point and thus to provide conditions suitable for continuous bacterial multiplication, this in turn resulting in more complete oxidation. Support is given to this theory by results of experiments in which physical and chemical means were used to reduce the bacterial population. Such reductions were invariably followed by renewed bacterial multiplication with accompanying oxidation. Four charts, 17 tables, and 11 references are given.—*R. E. Noble.*